Groundborne vibration caused by mechanised construction works

Prepared for Quality Services — Civil Engineering, Highways Agency

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Executive Summary

Environmental considerations have become an important aspect of all major construction schemes. This report provides data and advice against which objections to schemes may be judged and methods for predicting the environmental impact of vibration caused by the operation of mechanised construction plant.

Before this research was undertaken, the Specification for Highway Works only contained guidance on vibration arising from blasting works (Clause 607). There was a need for guidance on vibration from other construction activities and revision was required to incorporate the recommendations of BS 7385 Part 2 'Guide to damage levels from groundborne vibrations'. By reference to this Project Report, the Specification for Highways Works will be amended to address these issues.

Previous research into the effects of ground vibration undertaken at TRL has concentrated largely on blasting works. The research described in this Project Report has provided an essential follow-up to the earlier work. The topics covered by this Project Report are:

- a detailed review of the literature on ground vibrations from compaction, piling, tunnelling and other mechanised construction and ground improvement techniques;
- a review of national and European standards providing threshold values for damage and intrusion by groundborne vibration;
- the acquisition of field data from construction sites for most types of vibratory site operations;
- the execution of a full-scale trial to investigate groundborne vibration caused by vibratory compaction plant;
- analysis of the vibration data acquired from construction sites, the full-scale trial and other research;
- prediction of vibration from mechanised construction operations.

The proposed predictors allow the calculation of expected vibration levels of groundborne vibration for the following activities:

- vibrating rollers;
- vibratory piling, including vibrated casings for bored piles;
- percussive piling;
- dynamic compaction;
- mechanised tunnelling;
- vibratory ground treatment.

Probabilistic predictors are presented for vibratory compaction and vibratory piling. For the other activities, predictors are presented as upper bounds to the data and therefore are expected to be generally conservative. The predictors are generally valid at distances of up to about 100m from the vibration source. This encompasses the distances at which ground vibration is likely to be perceptible at most sites, although the effects of some operations may be perceptible at greater distances. Extrapolation much beyond this distance is not recommended although it will generally provide a conservative estimate.

1 Introduction

1.1 Objectives

The increasing size and power of construction plant and its potential to dissipate intrusive or possibly damaging levels of vibration into the environment, coupled with increasing attention being given to environmental aspects of road construction, have led to a need for improved methods of ground vibration prediction. While there is an increasing need to minimise the intrusive effects of construction works, over-conservative restrictions on vibration levels may lead to significant and unnecessary cost increases.

Previous research at TRL into the effects of ground vibration has concentrated largely on blasting works (New, 1986). Research elsewhere has largely focused on piling and, to a lesser extent, dynamic compaction. Vibratory compaction has received relatively little consideration. Consequently it is on this type of plant which the current research has predominantly focused. Consideration has also been given to percussive and vibratory piling; dynamic compaction and vibratory ground treatment; and mechanised tunnelling works. The application of the results will ease the control and mitigation of vibration to prevent damage or intrusion without the imposition of over-conservative limits, and aid selection of appropriate plant.

The main objective of the research was to provide a rational basis for guidance to be included in the Specification for Highway Works. To achieve this objective it was necessary to review research which had been previously undertaken, both at TRL and elsewhere, and to identify those areas requiring further study. The research aimed to present means of predicting vibration levels from all mechanised construction activities which may cause sufficient levels of groundborne vibration to be environmentally intrusive, given a knowledge of the characteristics of the plant and site conditions.

The potential for groundborne vibration to cause nuisance or damage may be made by comparing the predicted levels of vibration on a site with the guidance provided by the relevant national Standards. Section 2 of this report reviews the guidance on levels of vibration which are considered to be damaging and those which may cause disturbance given by current British and other national standards.

Validation of existing methods for the prediction of ground vibration levels, and the development of new predictors where none existed, required the determination of the levels of vibration arising from the diversity of plant in current use. A combination of measurements of vibration from plant operating on construction sites and, for vibratory rollers, on a more controlled trial site was used. Section 3 describes the processes of data acquisition. The details of the experimental work for the controlled trial are presented in Appendix A.

Sections 4 to 7 of the report each start by reviewing the available literature relevant to the generation and prediction of vibration from, respectively, vibratory compaction, piling, ground improvement works and mechanised tunnelling. The sections each continue by presentation and discussion of the new data acquired during the current research and conclude by making recommendations for predicting vibration levels. The concluding Section summarises the predictors presented within the document.

1.2 Specification of vibration

Comprehensive description of vibration requires details of the magnitude and frequency of the vibration, together with an indication of how these parameters change with time. Vibrations of physical systems can be decomposed by means of Fourier analysis into harmonic components, enabling ground vibrations to be fully defined by any pair of the inter-related parameters peak particle displacement, peak particle velocity (ppv), peak particle acceleration and frequency.

The capture of a complete time-history allows the determination of all the parameters required for a complete definition of the vibration signals. The definition of the parameters required to describe the magnitude of vibration in terms of ppv and the distance between the vibration source and measurement location are described below.

1.2.1 Magnitude of vibration

Groundborne vibration is commonly described in terms of peak particle velocity, because building damage has been shown to be well correlated with this parameter (New, 1986). Human sensitivity to vibration has also been shown to be constant in terms of peak particle velocity over the frequency range from 8Hz to 80Hz (British Standards Institution, 1992a) which covers the range of frequencies most commonly encountered from construction works. The ppv is also easy to measure, using moving-coil geophones, which have an output proportional to particle velocity. The peak particle velocity has been quoted in the literature in four different ways, being the peak value attained by:

- the vertical component (although this may not be the largest);
- the largest of the three mutually perpendicular components;
- the *true* resultant, which is the maximum value of the vector summation of the three components;
- the *pseudo*, *simulated* or *square root of the sum of squares* (*SRSS*) resultant of the three components, which is the vector sum of the maximum of each component regardless of the times at which the maxima occur.

The method of presentation of vibration data affects the quoted magnitude of vibration. The pseudo resultant is, by its definition, the maximum value which can be quoted and is usually an over-estimate of the true resultant The pseudo resultant could theoretically exceed the true resultant by a factor of $\sqrt{3}$, but is typically 20% higher than the true resultant, although this depends on the characteristics of the waveform (Hiller and Hope, 1998). The true resultant may not necessarily be significantly larger than the largest component, but if only a single value is to be quoted, then it errs on the side of caution to use the true resultant. Ideally, a vibration assessment should present the true resultant and the individual components. A number of

authors have reported data only as the magnitude of the vertical component; this is unsatisfactory since commonly one of the horizontal components is the largest of the three.

1.2.2 Distance terms

The definition of the distance from the source of vibration to the point of interest is an important parameter in the determination of prediction methods. The measurement of distance is important if results from different studies are to be compared. Where possible the current research has recorded both the horizontal and vertical distances between the sources of vibration and the measurement locations, so that the direct, or 'slope' distance could be calculated when appropriate.

When considering ground vibration from construction operations at the ground surface, the distance from the source to the point of interest is generally unambiguous. For tunnelling works, it is clearly important that the depth as well as the lateral distance to the activity is considered. In the case of piling, a significant amount of the energy transmission to the ground takes place at the pile tip. The distance may therefore be defined as that between the pile tip and the measurement location. This is used in some papers and is accurately described (eg Martin, 1980). Skipp (1984) and Uromeihy (1990) described this measurement as the *radial* distance, whilst Wiss (1967) and Mallard and Bastow (1979) used the term *seismic* distance and Gutowski *et al* (1977) used *vector* distance.

For ease of measurement and application on site, many authors have used the distance measured horizontally along the ground surface from the point where the pile enters the ground, regardless of the depth to which it has been driven. This distance is variously described as the *horizontal* (Li *et al*, 1990), *plan* (Skipp, 1984), *radial* (Oliver and Selby, 1991), or *standoff* (Uromeihy, 1990) distance. In some cases the way in which the distance has been measured has not been quoted (for example Jongmans, 1996; Massarsch, 1992). A number of reports use more than one term, for example Oliver and Selby (1991) use *stand-off*, *radial*, and *horizontal surface* distance. Attewell (1995) distinguished between the direct distance from the source, which in some cases may be the pile tip, and the horizontal radial distance.

1.2.3 Temporal variation of vibration

In addition to the magnitude and frequency of vibration, the potential for damage and disturbance is also dependent upon the nature and duration of the vibration. Ground vibrations can be classified as follows (Figure 1).

- *Continuous vibration*: cyclic variation in amplitude which repeats many times.
- *Transient vibration*: cyclic variation in amplitude which reaches a peak and then decays towards zero.
- *Intermittent vibration*: a sequence of transient vibrations with sufficient time between events for the amplitude to decay to an insignificant level.
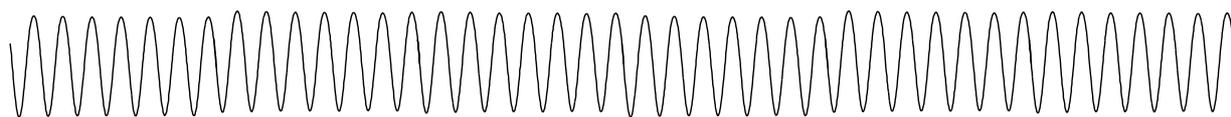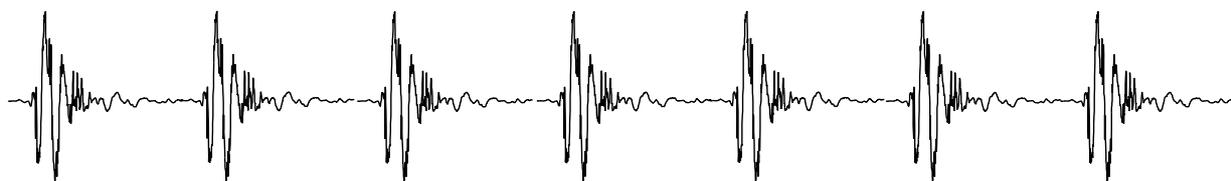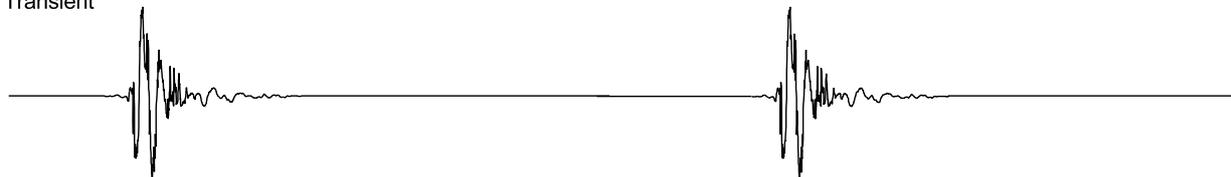
Figure 1 Temporal variation of vibration

● *Pseudo steady state*: a sequence of transient vibrations which are sufficiently closely spaced that the coda of each event overlaps with the arrival of following event.

In general, continuous vibrations are likely to be more damaging and intrusive than intermittent vibrations and therefore lower levels are permissible. These aspects are discussed in Section 2.1 in relation to guidance in British Standards.

1.3 Attenuation of ground vibration

Attenuation describes the processes by which the magnitude of a vibration reduces as it propagates away from the source. Attenuation occurs by two principal methods; geometric spreading and material damping. The first effect is purely geometrical and occurs because the energy radiated from the source is spread over an increasingly large volume of material as the wavefront propagates. This effect is, therefore, independent of the material properties of the ground. The second effect, material damping, is a property of the propagating medium and describes the frictional energy losses which occur during the passage of a wave. In addition, attenuation may be caused by spreading of the waveform through different propagation velocities of the different wave modes and by mode conversion, reflection and refraction at discontinuities. The relative importance of these effects and how, in practice, they have been accounted for are described below.

1.3.1 Theoretical considerations

Vibration is propagated away from the source in the form of *body* waves and *surface* waves. Body waves can be further subdivided into *compressional* waves and *distortional* or *shear* waves, the velocities of which are related by the Poisson's ratio of the soil:

$$\frac{c_p}{c_s} = \left[\frac{2(1-\nu)}{(1-2\nu)}\right]^{\frac{1}{2}} \tag{1}$$

where c_p is the compressional wave velocity of the soil;
c_s is the shear wave velocity;
ν is the Poisson's ratio.

For soils, the compressional wave velocity is typically about 1.7 times the shear wave velocity.

The most important type of surface waves in relation to construction induced groundborne vibration, Rayleigh waves, have a particle motion which is a retrograde ellipse perpendicular to the ground surface and an amplitude which decreases exponentially with depth. The propagation velocity of Rayleigh waves, c_r is related to the shear wave velocity by the expression:

$$c_r = Kc_s \tag{2}$$

where K is a function of the Poisson's ratio (Richart *et al*, 1970):

$$K^6 - 8K^4 + 8\left[3 - \frac{1-2\nu}{1-\nu}\right]K^2 + 16\left[\frac{1-2\nu}{2(1-\nu)} - 1\right] = 0 \tag{3}$$

Graff (1975) reported an approximate relation between K and ν

$$K = \frac{0.87 + 1.12\nu}{1+\nu} \tag{4}$$

The Rayleigh wave velocity is similar to, but slightly lower than the shear wave velocity. A second type of surface waves, Love waves, occur at the interface between two strata and appear only if the underlying layer has the greater shear wave propagation velocity.

In a purely elastic material, attenuation occurs only through geometric effects, caused by the increasing surface area (for body waves) or length (for surface waves) of the wavefront as the energy spreads away from the source. The energy attenuates according to $1/r^2$ for body waves and $1/r$ for surface waves, where r is the distance from a point source. Therefore the particle velocity attenuates according to $1/r$ and $1/r^{0.5}$ for body and surface waves, respectively. Geometric attenuation is independent of the properties of the material through which the energy propagates.

Miller and Pursey (1955) showed that approximately two thirds of the energy from a source located at the ground surface is radiated as Rayleigh waves. Combined with the lower rate of attenuation of surface waves than body waves, this indicates that it is Rayleigh waves which are the most significant when considering the effects of groundborne vibration from construction works.

A number of authors (for example Mintrop, 1911; Bornitz, 1931; Barkan, 1962; Richart *et al*, 1970) have reported that attenuation in soils is more rapid than that predicted by elastic theory and that even for small deformations caused by the propagation of seismic waves, the amplitude is also attenuated by material damping caused by frictional dissipation of the elastic energy. Geometric and frictional attenuation effects may be combined in an expression of the form presented by Mintrop (1911, cited by Bornitz, 1931);

$$A_1 = A_0 \left[\frac{r_0}{r_1}\right]^{\beta} e^{-\alpha(r_1 - r_0)} \tag{5}$$

where A_1 is the amplitude at distance r_1 from the source;
A_0 is the amplitude at distance r_0 from the source;
α is the absorption coefficient; and
β is the geometric spreading term.

Mintrop (1911) presented this equation with ß having a value of 0.5, implying that it was intended to describe the attenuation of surface waves. The range of distances with which civil engineering works are concerned is relatively small compared with geophysical seismic distances and typically only a few wavelengths of the disturbance occur between source and receiver. Jaeger and Cook (1976) considered that dissipative attenuation did not assume significance until a propagation distance of a few orders of

magnitude greater than the vibration wavelength. Consequently it is the geometric attenuation which dominates with the influence of internal friction being a secondary effect (Attewell, 1995). While accepting that theoretical geometric spreading models alone seriously under estimated the observed rate of attenuation of seismic waves, New (1984) argued that the additional attenuation was only in a small part due to material damping effects but was mostly attributable to other geometric loss mechanisms. In particular, it was observed that spreading of the wave packet, the duration of which increased linearly with propagation distance, would account for considerable attenuation of the measured particle velocity.

Massarsch (1992), however, considered that material damping does have a strong influence on the attenuation of ground vibrations and suggested that the omission of this effect may be one reason for the often poor correlation of vibration data. Massarsch presented curves showing the theoretical effect of material damping on the amplitude of Rayleigh waves which showed the amplitude at 50m from the source to be more than an order of magnitude higher without damping than when a coefficient of attenuation (α) of 0.05m^{-1} is assumed. For comparison, Greenwood and Farmer (1971) and Heckman and Hagerty (1978) assumed α to have a value of 0.03ft^{-1} (0.10m^{-1}). However, these authors considered that this value is applicable to all cases and is not dependent upon the geology. This factor could therefore be incorporated in a general attenuation relation combining both geometric and frictional effects. Wood and Theissen (1982) suggested that it may be possible to model satisfactorily vibration data both by models which include a material damping term and by those which do not, given the amount of scatter which is typically observed. The following section describes how some authors have approached this problem in practice.

1.3.2 Practical treatment of vibration data

A number of experimental studies have sought to assess the importance of material damping on vibration attenuation, for example Attewell and Ramana (1966), Portsmouth *et al* (1992), Sams *et al* (1997). Whilst it is accepted that, providing the wave has propagated for a sufficient number of wavelengths, the material damping is a property of the propagating medium, it is unclear from the literature whether the absorption coefficient has a frequency dependence when considering shorter transmission paths (Dym, 1976). Barkan (1962) reported that for a viscoelastic material, damping constants are proportional to the squares of their frequencies, whereas for elastic media, the attenuation is proportional to the first power of frequency. However, Barkan also presented experimental data from two tests in water saturated fine grained sands which showed that attenuation was essentially independent of frequency in the range 10 to 30Hz.

Wiss (1967) observed a greater rate of attenuation in clay than in sand but Attewell and Farmer (1973) considered that material damping could be ignored for all practical purposes. Attewell and Farmer determined that the peak particle velocity reduced according to $1/r^{0.87}$ which they approximated to $1/r$. While this apparently implies a body wave attenuation, particle trajectories presented by Attewell and Farmer suggested particle motions similar to Rayleigh waves. The index having a value greater than 0.5 implies that energy losses by mechanisms other than geometric spreading are also important. Such expressions indicate a greater rate of attenuation than would be expected by geometric spreading alone, but simplify the inclusion of other effects.

Measurements made by White and Mannering (1975) in London Clay and Barton Sand indicated a complex variation of attenuation with frequency. A minimum attenuation was observed in the region of 10-20Hz with a general increase with frequency in the range 20-50Hz and some increase at frequencies of 10Hz and below. Watts (1992) found no general trend of attenuation with frequency but reported that for tests on a variety of soils, a low frequency pass band was present. The centre frequency of this pass band was found to increase with increasing soil stiffness.

Gutowski and Dym (1976) considered the attenuation of vibration generated by road traffic. Data were plotted against the quotient of distance and wavelength, which would yield a straight line relation if attenuation was linearly related to frequency. However, it was found that a logarithmic relation gave a higher correlation. The authors suggested that damping by soils may be non-linear and may be greater where amplitudes are higher, close to the source, than for smaller amplitudes of vibration. An alternative hypothesis suggested that the difference in attenuation rate with distance was a result of stratification or inhomogeneities in the soil reflecting and scattering more energy back to the surface at greater distances.

The attenuation effects of different geological materials has been considered in research on piling induced vibration reported by Uromeihy (1990) and Attewell (1995). A number of indices describing the rate of attenuation of vibration from piling works were presented which, while many of the attenuation rates appear to be related to other parameters, the different soil types were also considered to have had an effect. Brenner and Chittikuladilok (1975) analysed their data depending upon which material the pile tip was penetrating. It was found that the attenuation rate for the surface layer was best described by $1/r^{1.5}$, whereas for deeper layers $1/r$ gave a better correlation. However, an analysis of variance revealed that all the data could be combined, and a relation $1/r^{0.85}$ was determined, which is very similar to the relation presented by Attewell and Farmer (1973).

Many authors have simplified the effect of differences in internal damping of different earth materials and presented attenuation relation which are dependent only upon distance from the source. In general, single parameter relations have been presented showing the rate of attenuation with distance to be constant, as illustrated by those cases described in the preceding paragraphs. However, O'Neill (1971) found that close to the pile the amplitude reduced according to an inverse square of the distance whereas at greater distances the amplitude varied inversely with distance. This supports the idea that body waves dominate close to the source and it is only at greater distances that surface waves become significant.

Conversely, Richart *et al* (1970) and Gutowski *et al* (1977) showed that extrapolation of far field data over-estimated near field vibration levels, which could imply that more rapid attenuation occurs at greater distances, which contradicts O'Neill's observation. Harris and Kirvida (1959) measured vibrations from a vibratory compactor operated at distances of between approximately 7 and 300m. The data showed that at distances up to approximately 50m the attenuation reduced according to $1/r$, whereas at greater distances an average value of approximately $1/r^2$ was observed *ie* the reverse of the results reported by O'Neill.

To summarise, attenuation rates for field data have generally been observed to be greater than those which could be attributed to geometric effects alone and a curvature in the data field has been reported by many authors. However, field data are often presented as a straight line relation on a log-log plot of ppv against distance, with an attenuation rate which is greater than the theoretical rate, implying that effects other than simple geometric attenuation are important (Figure 2). Within the range of the data, this is usually as good a fit as can be achieved by curve fitting but extrapolation beyond the data field can lead to significant errors. However, as Dowding (1996) noted, predictions from such extrapolation yield a conservative assessment of vibration and this, combined with the simplicity of the power law model, has led to the almost universal use of this approach.

2 Vibration thresholds for damage and intrusion

Problems caused by groundborne vibration may take one of three forms. The most severe cases of vibration may cause actual damage to existing structures. However, the two more common sources of complaint are direct vibration disturbance (*perceptible intrusion)* to occupants of buildings and *audible intrusion* due to groundborne noise being radiated from elements of a structure which are caused to vibrate. Intrusion is more common than damage because the levels of vibration which are perceptible are at least an order of magnitude smaller than those which may cause damage. Vibration impacts may therefore be classified according to whether the levels are sufficient to be damaging or merely intrusive.

If vibration is intrusive then classification of the severity of the disturbance is required. The level of *perceptible intrusion* is dependent not only on the magnitude of the vibration but also on other factors, particularly its duration. To classify intrusion, an appropriate measure of the average level of vibration a specified period may be used. The method currently preferred by the British Standards Institution is the a vibration dose value (VDV) (BSI, 1987; 1992a). There is however some debate as to the validity of vibration dose calculations. Brodowski (1990) considered the VDV to provide a significant advance in the assessment of intrusion, but Trevor-Jones (1993) and Jefferson (1998) identified a number of problems with the approach.

Audible intrusion due to groundborne noise requires separate treatment to allow classification in terms of
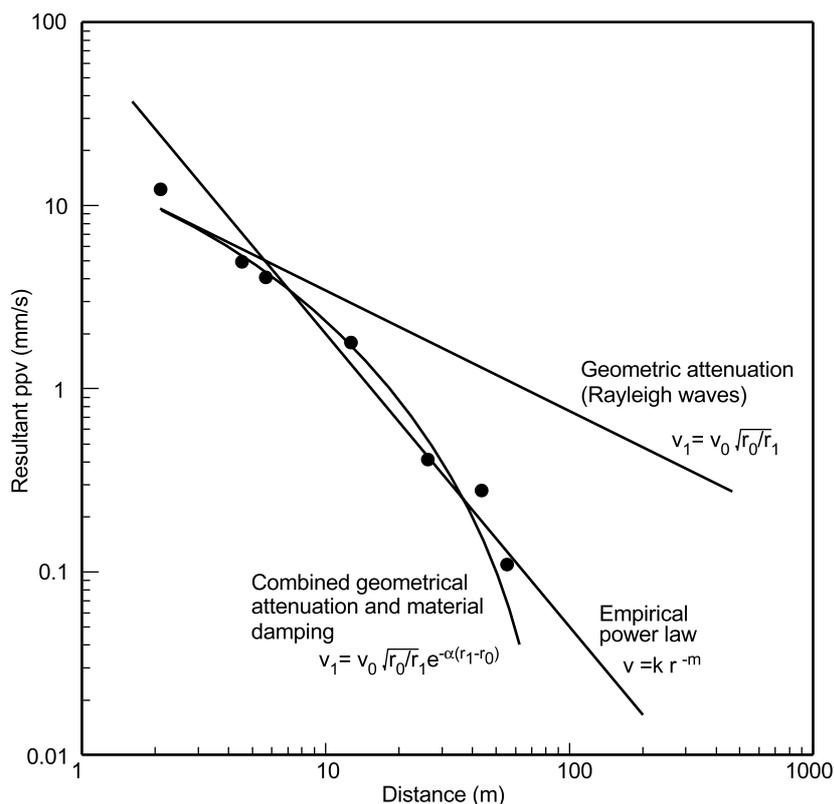

Figure 2 Graphical representation of attenuation (from Woods & Jedele, 1985)

established noise parameters. The vibration level required to cause intrusive groundborne noise is less than that required for direct perception and so can potentially be problematic over a wider area. Groundborne noise is, however, generally only likely to be a problem when there is no significant intrusion caused by airborne noise.

The following sections outline the parameters used to quantify vibration and briefly review the current British Standards which specify thresholds at which damage and disturbance may occur. Data presented and reviewed herein are largely presented in terms of peak particle velocity (ppv). This parameter has been found to be best correlated with case history data relating to building damage since particle velocity is proportional to the strain induced during the passage of a wave (New, 1986). Furthermore, the current relevant British Standards refer to vibration levels in terms of peak particle velocity, therefore the rest of this discussion will focus on this parameter.

2.1 British standards

2.1.1 Thresholds for damage to structures

There are currently two British Standards which offer advice on acceptable levels of vibrations in structures. BS 7385 : Part 1 : 1990, *Mechanical vibration and shock - vibration of buildings - guidelines for the measurement of vibrations and evaluation of their effects on buildings*, discusses the principles for carrying out vibration measurements and processing the data. Part 2 of the Standard, *Evaluation and measurement for vibration in buildings. Guide to damage levels from groundborne vibration* (BSI, 1993), suggests levels at which the following three categories of damage might occur.

Cosmetic   The formation of hairline cracks on drywall surfaces, or the growth of existing cracks in plaster or drywall surfaces; in addition, the formation of hairline cracks in mortar joints of brick/concrete block construction.

Minor   The formation of large cracks or loosening and falling of plaster or drywall surfaces, or cracks through bricks/concrete blocks.

Major   Damage to structural elements of the building, cracks in support columns, loosening of joints, splaying of masonry cracks, etc.

BS 7385 recommends that the peak particle velocity is used to quantify vibration and specifies damage criteria for frequencies within the range 1Hz to 1kHz. Limits for transient vibration, above which cosmetic damage could occur, are presented numerically and graphically for frequencies between 4Hz and 250Hz, which is the range usually encountered in buildings. These are reproduced in Figure 3. At frequencies below 4Hz it is recommended that a maximum displacement of 0.6mm (zero to peak) should be used. Minor damage is considered possible at vibration magnitudes which are twice those given and major damage to a building structure may occur at levels greater than four times those values. The Standard's guide values relate to transient vibrations and to low rise buildings. Continuous vibration can give rise to dynamic magnifications due to resonances. BS 7385 : Part 2 : 1993 states that the guide values may need to be reduced by up to 50 per cent for continuous vibration. However, it is noted that cases where continuous vibration has caused damage to buildings are too few to substantiate the guide values which are based on common practice.
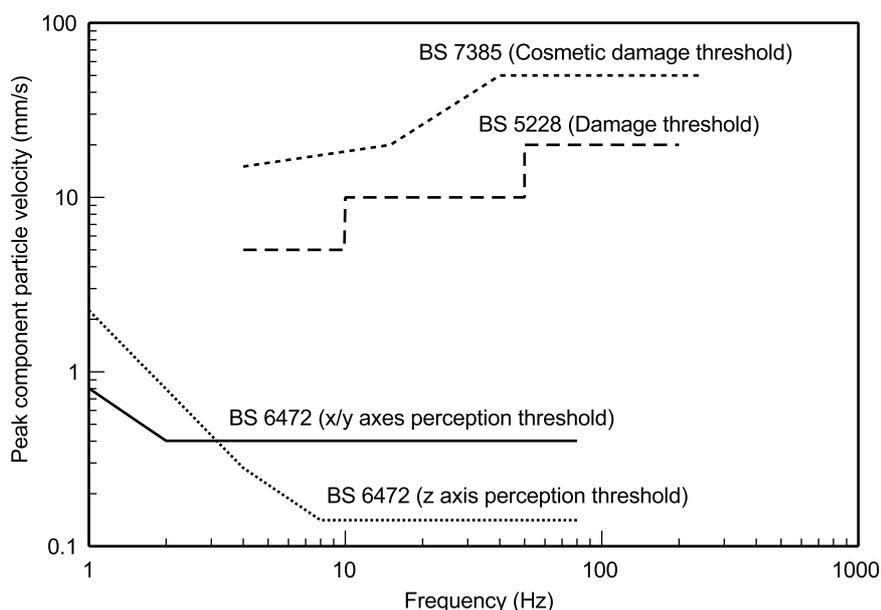

Figure 3 Thresholds provided by British Standards for human perception and damage to domestic structures by transient vibrations

Guidance on acceptable vibration levels in structures is also provided by BS 5228 : Part 4 : 1992, *Code of practice for noise and vibration control applicable to piling operations* (British Standards Institution, 1992b). This Standard recommends that a conservative threshold for minor or cosmetic damage should be taken as a peak particle velocity of 10mm/s for intermittent vibration and 5mm/s for continuous vibrations. It is recommended that these limits should be reduced by up to 50 per cent if buildings contain pre-existing defects of a structural nature. These limits are compared with those from BS 7385 in Figure 3. It is not clear why there is a discrepancy between the two Standards.

It is of interest to note that the use of resultant particle velocities is not specified in either of these Standards. Threshold levels are presented in terms of peak component particle velocities since the majority of data on which guide values have been based were reported in terms of peak component particle velocity (BSI, 1993). BS 7385 recommends that the peak true resultant particle velocity should be used for a detailed engineering analysis, for which the measuring directions should be specified.

2.1.2 Thresholds for human perception and disturbance
Assessment of disturbance by groundborne vibration must consider whether the magnitude of vibration is sufficient to be perceptible and, if so, whether the duration for which the vibration exists is likely to give rise to complaint. The combination of the magnitude and duration of vibration is quantified by the vibration dose, and its impact depends also upon the number of events, time of day and location of the recipient. Details of the currently accepted calculation procedure are presented by BS 6841 (BSI, 1987) and BS 6472 (BSI, 1992a). For brevity, the following discussion considers only the thresholds at which vibration is considered to be perceptible.

BS 6472 : 1992, *Evaluation of human exposure to vibration in buildings (1Hz to 80Hz)* (BSI, 1992a), provides guidance on human response to building vibration by the provision of a series of weighting curves and recommended exposure levels in the specified frequency range. Consideration is given to the different sensitivity of humans to up-and-down and side-to-side vibrations (Figure 4). BS 6472 recommends that measurements are made in terms of particle accelerations but curves are presented for both particle accelerations and velocities, the curves for velocities being calculated assuming sinusoidal motion. Measurement in acceleration terms conflicts with BS 7385 which requires measurement to be made in terms of particle velocity for assessment of damage potential.

Base curves are presented in BS 6472 which define thresholds below which 'adverse comments or complaints of vibration are rare'. For z-axis vibration, the threshold
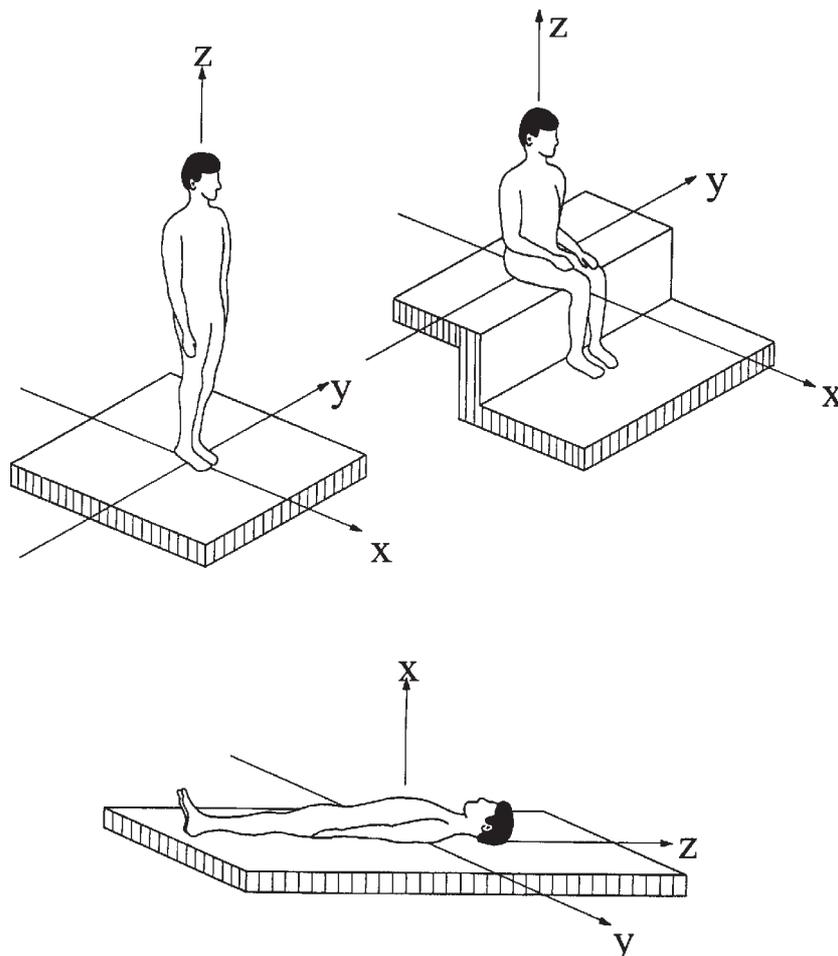

Figure 4 Co-ordinate system for human sensitivity to vibration (from British Standards Institution, 1992a)

level is at 0.141mm/s between 8Hz and 80Hz. The level is higher at lower frequencies, rising to 2.25mm/s at 1Hz. For x- or y-axis vibration, the curve is flat at 0.402mm/s over the frequency range 2Hz to 80Hz, rising to 0.804mm/s at 1Hz. The base curve values are applicable to the most sensitive situations. Multiplying factors are given in the standard to specify satisfactory magnitudes of vibration for different buildings.

The piling vibration standard, BS 5228 : Part 4 : 1992, also considers human response to vibration by reference to BS 6472, the threshold of perception being given as between 0.15mm/s and 0.3mm/s at frequencies between 8Hz and 80Hz.

BS 6611 : 1985, *Evaluation of the response of occupants of fixed structures, especially buildings and offshore structures, to low frequency horizontal motion (0.063Hz to 1Hz)* (British Standards Institution, 1985), provides guidance to the evaluation of the response of the occupants of buildings to vibrations at frequencies lower than those covered by BS 6472. Vertical vibration is not considered since the Standard was produced primarily to address the behaviour of tall buildings and off-shore installations exposed to wind and/or wave loading. The frequencies covered by this standard are below those normally encountered in civil engineering works and are also below those which the large majority of proprietary equipment is capable of measuring. Consequently this will be given no further consideration.

2.1.3 Thresholds for groundborne noise
There are currently no British Standards which cover human sensitivity to groundborne noise.

2.2 Other national standards
Detailed reviews of vibration standards in use in countries outside the United Kingdom have been given by Broadhurst *et al* (1984), New (1986), Attewell (1995) and Skipp (1998). This section of this report summarises the information presented by these authors and supplements and updates it where this has been possible.

2.2.1 Thresholds for damage to structures
Part 3 of the German standard (DIN 4150) relates to the effects of vibration on structures. This was updated in 1986 from the 1975 version to give a more detailed specification in terms of the frequency dependency of acceptability criteria and also to present limits in terms of individual components of vibration. The earlier version had presented limits in terms of resultant peak particle velocities. It is not clear why the change was made, but it is worth noting that the British Standard (BSI, 1993) also uses individual component values as discussed above. In contrast to the BS, DIN 4150 Part 3 gives guidance in terms of vibration levels recorded within the structure, which may partially account for the apparently more conservative limits. Reference values are quoted for short term vibration for the foundation and for the plane of the floor (horizontal vibration) of the uppermost full storey of the building. For continuous vibration, it is considered that

horizontal vibration measured at the uppermost storey up to 5mm/s for the whole structure and 10mm/s for building components is acceptable. The guidance given is appropriate for frequencies of up to 100Hz.

The French Standard (1986) gives considerable detail categorising a wide range of structures and presents three curves giving different limits depending upon the structure. Two sets of curves are given, covering continuous and transient vibrations, which refer to the vibration limits experienced by elements of the structure. In both cases, details are given from 4 to 100Hz, with higher (unspecified) levels allowable at higher frequencies. The requirements are more conservative than those given by the British and German standards, but similar to those given by Swiss Standard SN 640 312 1978.

Swedish limits are presented by Persson *et al* (1980). The quoted guidance values are only applicable to structures founded on hard rock and exposed to vibration caused by blasting but limits are the least stringent of all standards reviewed herein, although the Indian Standards Institution (1973) allowed similarly high levels.

Vibration limits specified in Finland (Vuolio, 1990) relate to blasting vibration and are therefore higher than those which might be considered acceptable for continuous vibrations. The limits are related to the dynamic ground strains generated; higher levels are acceptable on rocks of higher wave propagation velocity. The threshold values on three categories of rocks quoted correspond to approximately 20 to 30 microstrain.

The Austrian standard, reviewed by Attewell (1995) and the guidelines given by the Standards Association of Australia (1993) are similar in that the guide values are independent of frequency, however, the Austrian standard is rather more stringent. The values given by the Australian standard relate to the use of explosives, and are therefore concerned with transient vibrations. The limiting ppvs are suggested based on both structural integrity and by consideration of human discomfort. Logan and Sutherland (1997) reported that guidance in New Zealand is given in NZS 4403:1976. However, since the detail in NZS 4403 is restricted, the relevant Australian and German standards are often used.

The American National Standard (Acoustical Society of America, 1990) adopted a different approach and one which is not readily compared with those described above. It requires that measurements are made on components of structures of interest and the data converted to dynamic stresses and related in structural terms to allowable stresses. For prediction work, allowable stresses for structures would need to be converted to vibration level.

Eurocode 3, Chapter 5 (CEN, 1998) presents the most recently published guidance on vibration damage to structures. This document recommends the same thresholds as are given by the British Standard relating to piling vibrations (BSI, 1992b) and are therefore rather conservative. The conservatism is acknowledged, however, by a footnote which states that imposition of these limits would result in a low probability of even minor cosmetic damage occurring. Tabulated thresholds are given for five different types of structure.

The guidance given by the British and other standards is summarised in Figure 5. There is clearly a considerable difference between the levels of vibration that are considered to be acceptable in different countries. While this is undoubtedly partially due to political influence, there may also exist significant differences in ground type and construction methods which may affect the tolerance of structures to vibration. New (1986) reported that, in general, the more recent the standard the more conservative were the specified vibration limits. The British Standard BS 7385 : Part 2 (British Standards Institution, 1993) reversed this trend but the European guidance on vibration from piling (CEN, 1998) has reverted to more conservative guidance.

2.2.2 Thresholds for human perception and disturbance

It is more difficult to compare standards giving guidance on human tolerance levels than for acceptable levels of vibration in structures because of the different physiological sensitivity depending upon the direction of vibration and the variation of tolerance depending upon the location. For example higher levels of vibration are

tolerated in workshops than in critical working areas such as hospital operating theatres. Therefore, the following discussion compares recommended peak particle velocities for z-axis (ie parallel to the spine) vibration during the daytime (typically 07:00 to 23:00) within residential buildings. Figures for the *base curves*, *ie* the threshold of perception, are also compared.

The foreword to the British Standard on the evaluation of human perception to vibration (BSI, 1992) states that the International Standard, ISO 2631 : Part 2 (ISO, 1989), on the same subject does not contain sufficient information to enable a proper evaluation. The British Standard therefore updates the International Standard. However, the earlier document is still used in some countries, such as Australia (Standards Association of Australia, 1990) and New Zealand (NZS/ISO 2631:1992).

The Acoustical Society of America (ASA) defines thresholds for disturbance by vibration of the occupants of buildings (ASA, 1983) based on guidance given on reaction of humans to vibration transmitted to the human body as a whole (ASA, 1979). Thresholds are specified as root mean square (rms) velocities. Assuming sinusoidal vibration, the
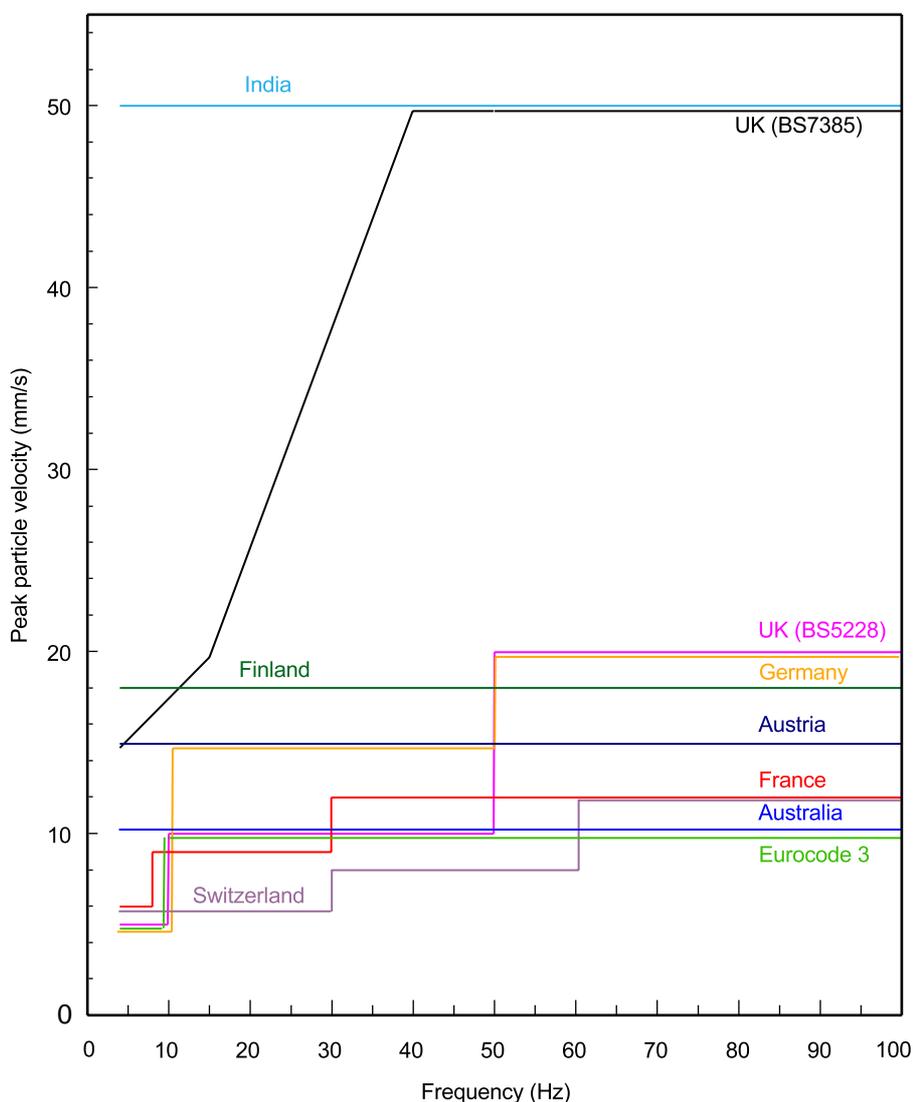

Figure 5 Summary of damage thresholds for transient vibration on domestic structures

thresholds are the same magnitude as those specified by BS 6472 (British Standards Institution, 1992a), which are quoted as peak values. Table 1 compares the threshold values for intrusion given by the British, American and Australian standardising authorities. The base curve values are all similar but the acceptable limits within residential properties differ, being lowest in America.

Eurocode 3, Chapter 5 (CEN, 1998) adopts a different approach to that given by other standards. Rather than presenting a disturbance threshold, EC3 recognises that tolerance of vibration is dependent upon the duration as well as the magnitude. The approach is similar to the vibration dose value method included in the British Standard (BSI, 1992a). However, the vibration dose value (VDV) calculated by the British Standard is based on a fourth power relation between particle acceleration and duration (*ie* a two-fold change in magnitude is equivalent to a 16-fold change in duration) based on research by Griffin (1990). The Eurocode is based on a square relation (*ie* a two-fold change in magnitude is equivalent to a four-fold change in duration). A threshold for perception is implied by the Eurocode since it is stated that, in particularly sensitive locations, vibrations up to 0.15mm/s should be acceptable.

2.2.3 Thresholds for groundborne noise
The American Public Transit Association (APTA) has published guidance on acceptable levels of groundborne noise in buildings based on vibration from trains. This guidance is summarised in Table 2. Groundborne noise below the levels given in this table should not cause significant intrusion or annoyance (APTA, 1981).

3 Data acquisition and reduction

To enable the validation and development of methods of predicting groundborne vibration levels, it was necessary to acquire and analyse data from the various construction activities which were to be considered. This section describes the experimental work undertaken for the research, including details of the equipment used and the methods of data acquisition. Most of the methodology described in this section is common to all of the types of construction activities which were investigated.

Table 2 APTA guidelines for maximum groundborne noise inside habitable premises

	Maximum groundborne noise level design goal (dB(A))		
Community area	Single family dwelling	Multi family dwelling	Hotel /motel buildings
Low density residential	30	35	40
Average density residential	35	40	45
High density residential	35	40	45
Commercial	40	45	50
Industrial/highway	40	45	55

3.1 The data acquisition equipment and its use

3.1.1 Instruments
The instrumentation system used for acquisition of field data both on live construction sites and during the controlled experiment (described in Appendix A) is illustrated in Plate 1 and, schematically, in Figure 6. A four wheel drive mobile laboratory (Plate 2) was used to transport the equipment, from which it was operated.

The transducers used for the majority of measurements were Sensor *SM6a* geophones with a natural frequency of 4.5Hz and a linear response (within ±5 per cent) between 5Hz and 300Hz. The zero-to-peak displacement limit of these geophones is 2mm. Geophones have the advantages of being self-generating and giving a high output, allowing use over very long lines. They also give an output voltage proportional to velocity, which is the most appropriate vibration parameter for the quantification of environmental vibration. For vibrations with amplitudes or frequencies outside the operating limits of the geophones, Monitran *MTN1100/75* accelerometers were used. These have a range of ±50g and a near-linear response proportional to acceleration, from about 1Hz to 10,000Hz, but are not suitable for low-frequency measurements when the outputs are integrated to give velocity. Further disadvantages of the accelerometer system are that accelerometers require a special power supply and are more susceptible to noise pickup than geophones. Within this research, use of accelerometers was largely restricted to measurements made directly on the drum of vibrating rollers.

Individual transducers were screwed into three orthogonal faces of aluminium cubes to create triaxial arrays which were used at each measurement location (Plate 3). Transducers were connected to the rest of the system via screened cables with rugged waterproof

Table 1 Comparison of threshold values for human perception recommended by various national standards

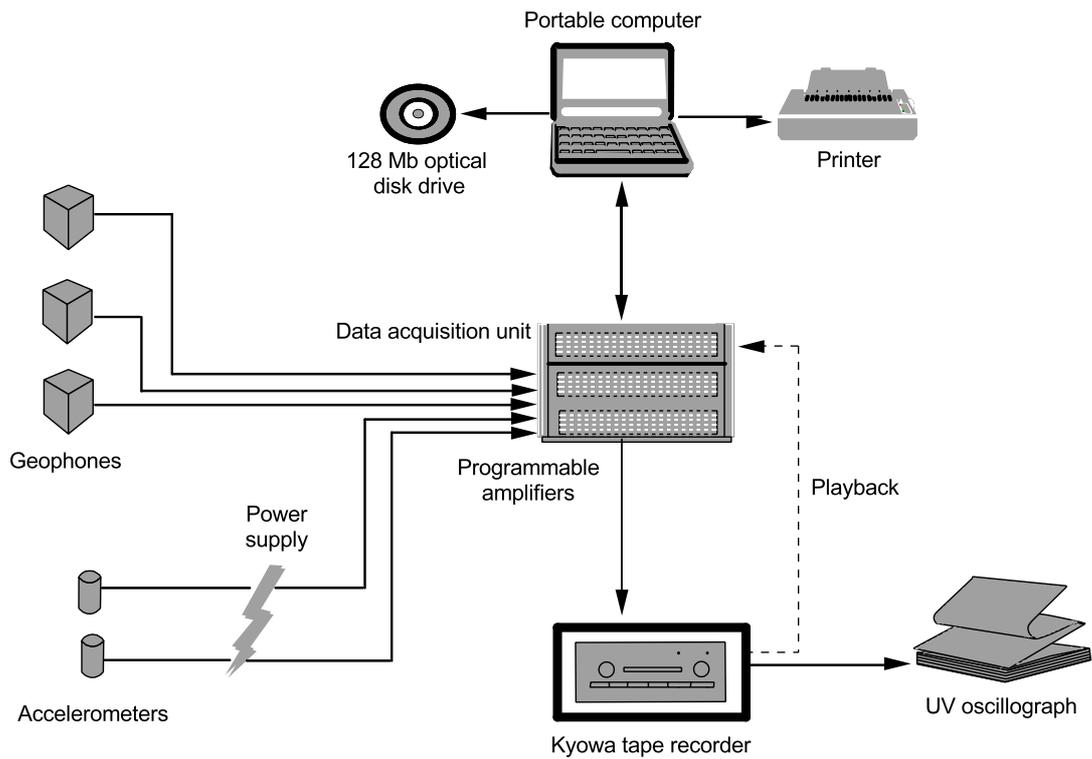|		Base curve particle velocity (mm/s)		Lower limits for residential property (mm/s)	
Country	Standard	x/y axes (at 2 to 80Hz)	z axis (at 8 to 80Hz)	Day	Night
UK	BS 6472 : 1992	0.402 (peak)	0.141 (peak)	0.804 to 1.608	0.563
USA	ANSI S3.29 - 1983	0.290 (rms)	0.100 (rms)	0.140 to 0.400	0.1 to 0.14
Australia	AS 2670.2 - 1990 (ISO 2631/2 - 1989)	0.287 (rms)	0.0995 (rms)	0.574 to 1.148	0.402

Plate 1 Monitoring equipment

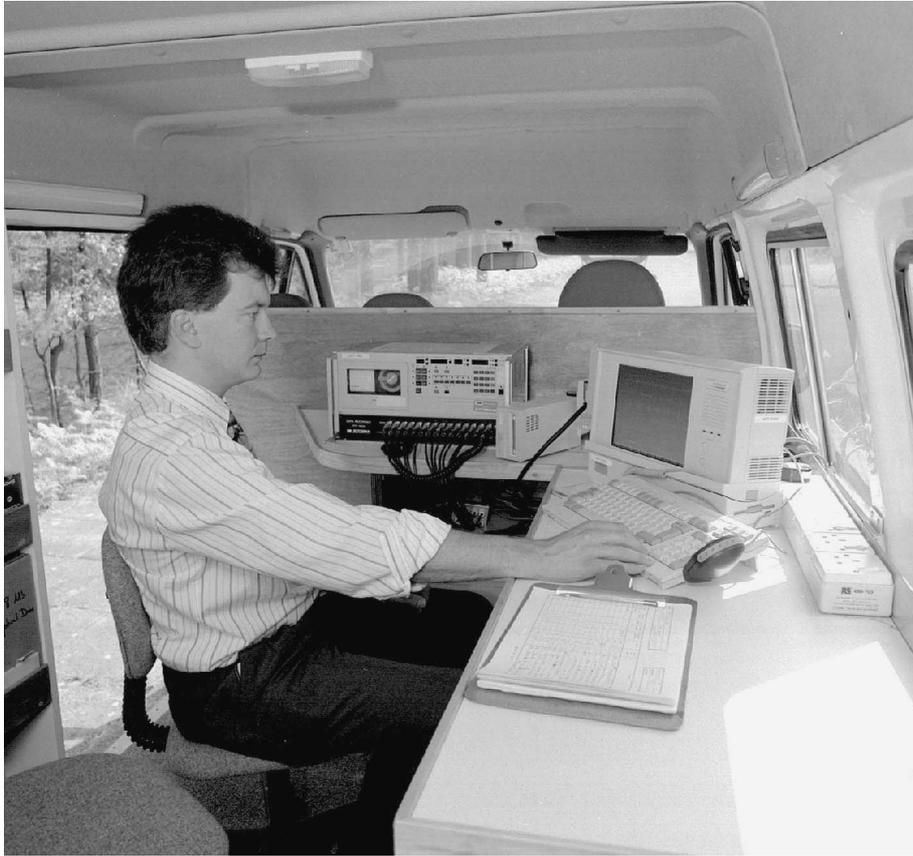**Figure 6** Schematic of data acquisition equipment
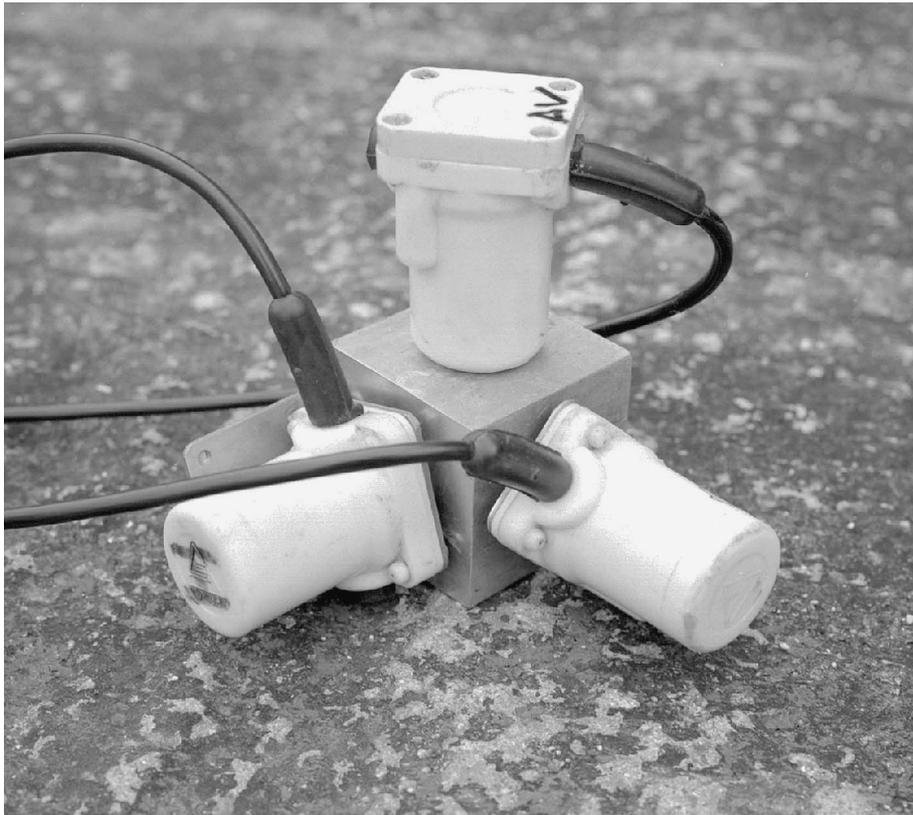
Plate 2 Mobile laboratory

Plate 3 Three geophones mounted orthogonally on a cube

connectors, enabling reliable use even in wet conditions. Analogue signals generated by the transducers were amplified and filtered, using Axxon *Cyberamps*, before being digitised and stored into memory by a Cambridge Electronic Design *1401plus* 16 channel data acquisition unit. The analogue-to-digital converter (ADC) in this system allows sampling at an aggregate frequency of up to 148kHz and has 16 Mbytes of RAM. At a typical operating frequency of approximately 2kHz, this enables up to four minutes of continuous signal to be captured. The amplifiers have a range from unity to 20,000 times amplification, which gives the *1401plus* a range of full scale deflections from 0.01 to 200mm/s when used with the geophones. The 12-bit ADC gives a resolution of 1/2048 of full scale. Digital data files were saved on 128 Mbyte magneto-optical discs for subsequent analysis.

Where continuous sampling was required for longer periods of time than was possible with the digital system, such as for monitoring tunnel excavation sequences, a Kyowa *RTP-400a* analogue tape recorder was used. This enables continuous sampling and storage on *Betamax* video tape of up to 14 channels of data, for periods in excess of an hour. The tape recorder was used in parallel with the digital system. During the data processing (Section 3.2.1), the tape recorded signals were replayed into an ultra-violet chart recorder and the chart was compared with the digital data. If the digital records had not captured the largest events, the appropriate sections of the tape were replayed into the digital system for further analysis.

3.1.2 Calibration of the instrumentation
To ensure the integrity of the data, all components of the acquisition system were calibrated before use. The amplification and recording components of the equipment, such as the digital data acquisition unit, the amplifier system and the tape recorder were all calibrated by their manufacturers. The accelerometers were also supplied with individual calibration certificates, but the geophones were specified only as having a nominal sensitivity for the type of geophone, not for individual instruments.

One accelerometer, for which a calibration curve was obtained from the manufacturer, was used as a reference accelerometer against which the calibrations of the geophones were checked, according to British Standard BS 6955 (BSI, 1994). The accelerometer was mounted back-to-back with each geophone in turn on an assembly mounted on an electrodynamic vibration generator . The signals from the transducers were captured by the acquisition system and the peak particle velocity (ppv) measured by each transducer was compared at frequencies of 15Hz and 70Hz. One vertical and two horizontal geophones were also tested over a range of frequencies from 4 to 300Hz to establish the frequency response curve for comparison with that supplied by the manufacturer (Figure 7). The sensitivity of each geophone was determined to be within the manufacturer's specified tolerance.

The response of geophones is directional and the sensitivity is specified within a limited range of angles of deviation about the axis. As a part of the calibration procedure, the effect on the sensitivity of the geophones to
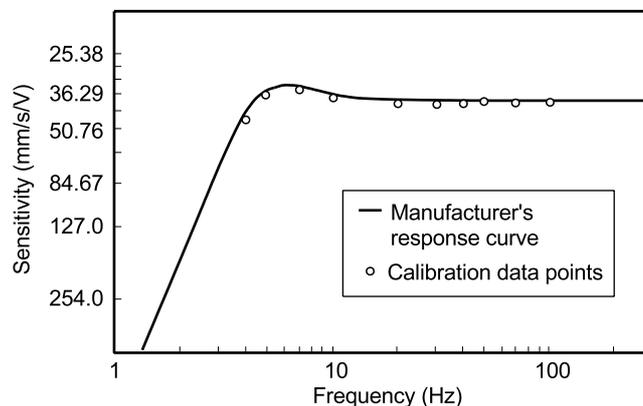
Figure 7 Measured geophone response curve

misalignment was investigated, to assess whether it was necessary to measure the mounting alignment. This revealed that the deviation in sensitivity increased to 1 per cent at ±4° tilt for a horizontal geophone and at ±13° for a vertical axis geophone. It was therefore concluded that alignment by eye would cause acceptably small errors.

3.1.3 Methodology for field measurement of vibration
Three orthogonal components of vibration were recorded at almost all measurement locations throughout this research. This was achieved by assembling triaxial transducer arrays from uniaxial transducers screwed into aluminium cubes (Plate 3). These arrays were then screwed on to 200mm long stainless steel spikes driven fully into the ground. Where necessary, any loose soil or vegetation was removed before the spike was driven. For the controlled experiment, for which the geophones were to remain in place for several weeks, geophone arrays were located in an excavation which was of a depth such that the top of the vertical axis geophone was on the same level as the ground surface. Each excavation was then backfilled with the excavated soil which was hand-tamped around the array to ensure good coupling with the ground and to minimise the risk of disturbance.

The number of locations at which vibration measurements could be made simultaneously was restricted by the data acquisition system. The attenuation of vibration with distance is approximately logarithmic (Section 1.3) and, conventionally, data are plotted on log-log axes. The geophones were therefore positioned with increasing separations between adjacent arrays as the distance from the source of vibration increased. Typically, geophones were positioned as close as practicable to the source, and then at distances up to approximately 100m from this point, although actual locations were occasionally dictated by the conditions on the site.

Groundborne vibration from mechanised construction works typically contains energy at frequencies no greater than 100Hz, and in most cases the dominant frequencies are much less than this. The analogue signal from each geophone was sampled at approximately 2kHz with a low-pass (anti-aliasing) filter at 800Hz. This ensured that, for all activities, the Nyquist frequency was well in excess of

the frequencies anticipated, so that all waveforms were accurately recorded. In fact the anti-aliasing filters were largely superfluous in this study because there was negligible energy in the signals at frequencies above the filter cut-off frequency.

On each site, and for each item of plant studied, attempts were made to acquire sufficient data from which to derive results which were statistically valid. In practice the amount of data was to some extent limited by the site operations and the time available.

3.2 Data reduction

The first stage of the analysis was to reduce the digital files to data of a form which could be analysed statistically, so that the various parameters which may affect the vibration level could be investigated. The data were principally interpreted in terms of the peak particle velocity (ppv), since this is the parameter used by the majority of national standards to quantify the potential of vibration to cause damage or intrusion (Section 2).

The method used for the first stage of processing was similar for all data and is described in Section 3.2.1. The analysis and interpretation of the data from each type of construction activity are described in subsequent sections.

3.2.1 Initial analysis and collation of data

The digital time histories were inspected to ensure that there were no spurious events within the records. Numerical data were then calculated from the time histories so that statistical analyses of the data could be undertaken. This section describes the methods used which were, in general, common to the analysis of data from all the construction activities studied.

The data were analysed using a suite of purpose-written software routines, mainly in the *Matlab* script language. The sequence of operations for each set of data was as follows:

1 Manual selection of the required length of signal from a graphical display of the whole sample. This enabled specific events within each file to be analysed separately, such as the starting transient, continuous operation and stopping transient for vibrating rollers (Figure 8).

2 Calculation of the component peak particle velocity from each transducer and the true resultant ppv, from each triaxial array of geophones.

3 Calculation of the root mean quad (rmq) ppv and the root mean square (rms) ppv in one-third octave frequency bands for all three orthogonal components.

4 The starting time, relative to the start of the file stored in the field, and the length of the processed section of signal, were tabulated within the output files.

5 The tabulated data were exported to a proprietary spreadsheet programme, where additional experimental details were added, such as the plant type, operation, and the distance between the vibration source and the monitoring position.

The spectral analysis undertaken in stage 3 above, to determine the one third octave frequency band data, required the following steps. Firstly, for each signal, the mean of the particle velocity at each sampled point over the whole sampled length was calculated and the zero level for the data was reset to this value. This step eliminated any zero offset in the data arising from instrument drift prior to acquisition and, in addition to being required for the spectral analysis, was necessary to ensure that the ppv data were calculated correctly. The data were then resampled to halve the effective sampling frequency. The sampling frequency of just over 2kHz used during acquisition had been sufficiently high that this was possible without losing any information. This process doubled the resolution of the spectra to approximately 0.5Hz, which improved the quality of the data at low frequencies.

Spectral analysis used a 2048 point fast Fourier transform (FFT) to which a Hanning window was applied. This FFT length enabled determination of the spectrum for a data slice of approximately two seconds of the resampled signal, which in most cases was insufficient to enable spectral analysis the required length of signal. A series of windowed slices, which were overlapped by 50 per cent of their length, were used to analyse the required signal length. The root mean square (rms) particle velocity in each one third octave band for each slice of signal was calculated. The maximum rms particle velocity for each frequency band was then stored. The maximum value was used rather than the mean because, where signals from discontinuous events, such as percussive piling, were processed, the mean value could be significantly reduced by including sections which consisted largely of the quiescent period between events. It was desirable to use a consistent approach for all processing.

The procedure described above was used to generate a separate spreadsheet for each live construction site and for each piece of plant used on each fill during the controlled compaction experiment. Further data were added to each spreadsheet relating to the characteristics of the plant, the geology and the fill type, as appropriate, together with the distance from the vibration source at which the vibration was measured. These spreadsheets were than used to investigate the factors which affect the level of vibration generated by the various construction activities, as described in the following sections.

4 Groundborne vibration from vibratory compaction

4.1 Review of previous studies

The benefits of vibration for improving the performance of compaction equipment have been exploited for many years. While the effects of vibration on compaction have been the subject of considerable study (see Parsons, 1992), relatively little attention has been given to the vibration which is radiated into the environment by vibratory compaction plant. The few cases which have been presented are largely limited in detail and site specific (for example Forssblad, 1965). A review of vibratory
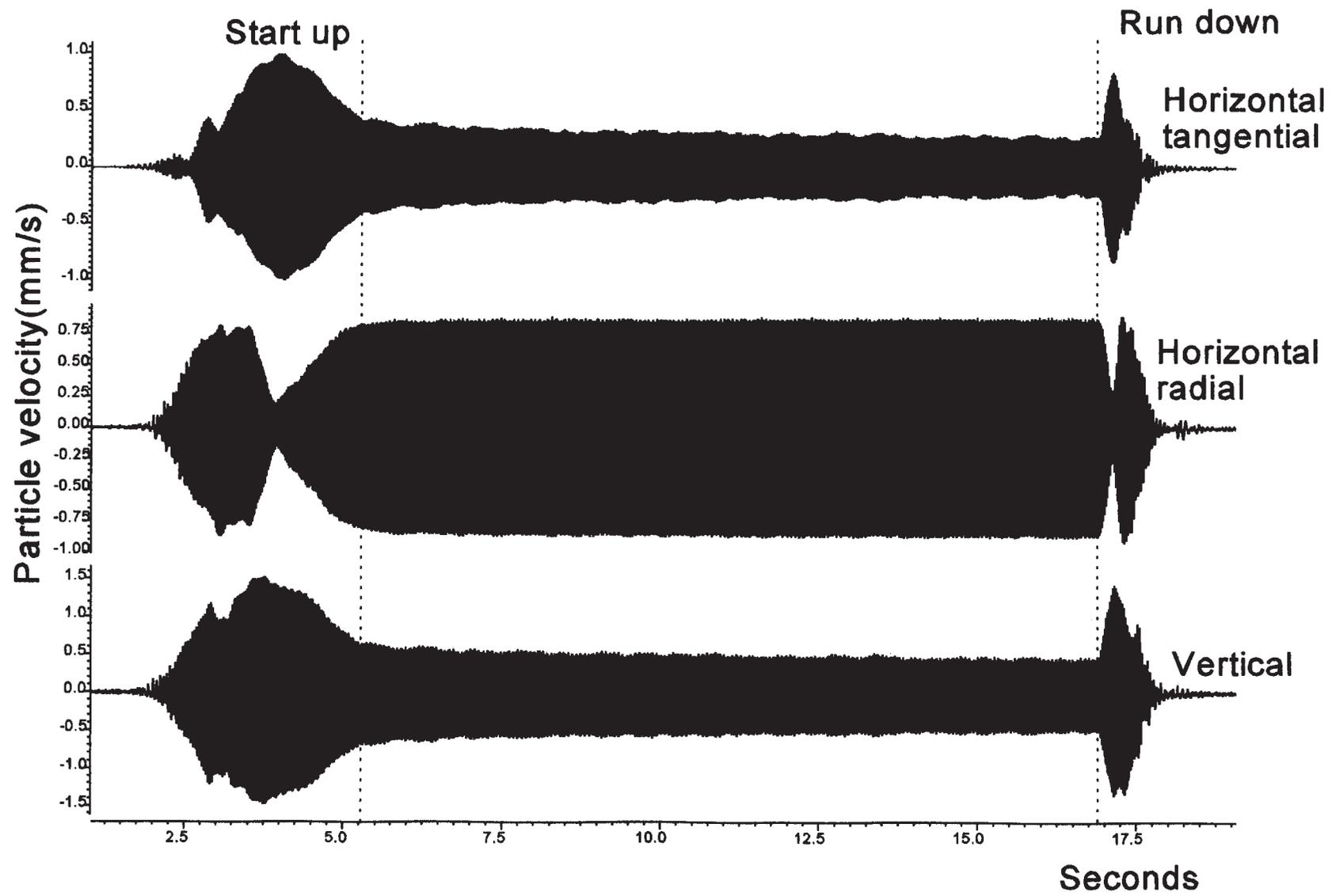
Figure 8 Signal from a vibrating roller, showing startup, steady state operation and stopping

compaction was undertaken by van der Merwe (1984) which limited discussion of the effects of off-site vibration to commenting on the experiments undertaken by Tiedemann (1970) which used only one sheepsfoot roller. Van der Merwe suggested that risk of damage should be considered if the roller operates within 3m of a structure: this is twice the distance suggested by Tiedemann.

Forssblad (1974) reported research undertaken by Appeltofft *et al* (1970) and a series of later tests in which the groundborne vibration from a number of vibrating rollers of different weights were measured. Forssblad concluded that the ppv could be predicted on the basis of the static weight of the roller. Forssblad observed higher levels of vibration during the starting and stopping of the vibrator than those generated during steady state operation. These transient high levels of vibration would limit the distance from property at which rollers could be used without the risk of damage. Based on a risk limit, for what Forssblad described as 'architectural' damage, of a peak particle velocity of 5mm/s it was proposed that the safe working distance for towed and self propelled vibratory rollers with pneumatic drive wheels, operating on soil, was

Safe distance in metres = 1.5 x drum module weight in tons

Similarly, for vibratory tandem rollers operating on soil and asphalt the expression given was

Safe limit in metres = 1.0 x drum module weight in tons

where drum module weight is defined as the static weight of the drum plus the frame weight transmitted to the drum. This predictor is of limited use since only one situation is considered; it is not possible to adapt the predictors to other specified limits such as consideration of the effects on other types of structures or for the assessment of intrusion.

Forssblad(1981) observed resonances associated with the starting and stopping of the drum which could generate higher levels of vibration than were caused by steady state operation. However, it is not clear whether this is accounted for in the expressions given above.

Wheeler (1990) studied the attenuation of vibration from rollers on ten construction sites, incorporating 15 combinations of roller type, operating frequency and ground conditions. A prediction equation was determined which had the same format as that used by Attewell and Farmer (1973) for piling induced vibration (Equation (14)), with *v* being the peak *resultant* particle velocity (in mm/s) and W_0 the theoretical energy per cycle (in joules). The latter parameter was determined from the quotient of the engine output and the vibrator operating frequency. The energy value used by Wheeler was the *output* (as specified by the manufacturer Stothert and Pitt) or *performance* (specified by Bomag). This would seem to be inappropriate since these values relate to the plant's engine. For a self propelled roller, some of this energy is required to drive the vehicle, whereas a towed roller can use all the available energy for the vibrator. It might therefore be expected that the vibration produced by a self propelled roller would be less than that from a towed roller with the same engine output. However, use of the energy

per cycle figure may reduce the scatter in the data since it must relate to the general size of the piece of plant and, in general, a large roller might be expected to give rise to higher levels of groundborne vibration than would a smaller roller.

Wheeler's data were acquired over a limited range of distances, up to 20m from the source, which in practice restricts the use of the predictor to damage assessment; for most rollers, vibration would be perceptible at distances far greater than this. Wheeler plotted the vibration field data using the same approach as Attewell and Farmer (1973) used for piling works: ppv against scaled distance (Section 5.2). An upper bound to Wheeler's data yielded a value of 3.16 for *k* (see Equation 14) for the true resultant ppv.

In comparison with the expressions proposed for piling works discussed below in Section 5, Wheeler predicted a significantly higher vibration level for a given nominal energy input. He suggested that this is due to a greater proportion of the energy being transmitted as surface waves than occurs from piling. Wheeler's data are assumed to be for steady state operation of the roller since Wheeler did not make reference to the transients which occur during starting and stopping of the plant. The actual levels may therefore exceed this prediction during the transient phases.

Dowding (1996) has suggested that the energy transferred to the ground from vibrating rollers can be calculated from a function of the machine weight, eccentric weight, the distance of the centre of mass of the eccentric weight from the centre of rotation and an amplification factor. Routine application of this method would be impractical because details of the eccentric are generally not quoted by manufacturers.

The information which is available in the literature is therefore of limited use for providing reliable prediction of groundborne vibration from vibratory rollers. In particular, little consideration appears to have been given to the many variables involved, and the range of distances over which data have been acquired is insufficient when considering the potential for intrusion. This has stimulated the current research for which a significant volume of new data has been acquired. The acquisition, analysis and interpretation of these data are discussed in the following sections.

4.2 Acquisition of vibration data

The review of the literature presented in Section 4.1 showed that, although vibratory compaction plant can give rise to levels of vibration which are potentially disturbing over large areas adjacent to construction works, no satisfactory method of predicting levels of vibration from such plant is available. This problem has been addressed by this Report. Vibration levels were recorded on a number of live construction sites and from operation of plant on a pilot scale test facility. These data were then used for the development of an empirical predictor of vibration. The two phases of data acquisition are described in the following sections.

The initial approach to the acquisition of data was to measure the levels of vibration arising from plant operating

on live construction sites. The rationale for this was that data from a range of plant operating on different geological formations would be acquired. Although the sites studied were all on road construction schemes, the data would be equally valid for predicting vibration from other constructions such as those for railways or earth dams. A total of 17 sites were visited on which measurements were made from a total of 11 different types of roller. These are summarised in Table 3 and Figure 9. The measurements made on live construction sites were intended to allow both the generation of vibration and its attenuation through different geological materials to be investigated.

At each site, the instrumentation was used as described in Section 3.1 to measure vibration as the works proceeded. The measurements were carried out such that construction would not be interrupted in any way which might give the Contractor justification for lodging a claim with the Engineer. Consequently there was little control possible over the activities monitored, although the plant operatives generally cooperated with the research requirements where possible. This restriction resulted in variable amounts of data being acquired at each site. The distance to the source was measured and this, together with any other relevant information, such as the presence of other plant working in the vicinity, was recorded.

Details of the geology and type of fill being compacted were supplied by the Resident Engineer at each site. Geological information was provided in the form of the site investigation data, comprising borehole logs and trial pit records. Fill types were classified according to the Specification for Highway Works Series 600, Table 6/1 (MCHW1, 1993).

While the acquisition of data from most activities on construction sites was successful, it became apparent that this approach was not the best means by which to acquire data from vibratory compaction. The following problems were encountered:

1 limitation in the quantity of data which could be obtained since hindrance of site operations was not acceptable;

2 no control of the speed at which the roller passed the monitoring locations;

3 difficulty in determining the number of passes of the roller over each section of the fill;

4 occasional uncertainty in the exact distances between the roller and the geophones because of the difficulty in identifying the location of the roller during the measurements;

5 potential contamination of the vibration records with vibration from other plant movements;

6 inconsistency in site topography;

7 variation between sites in the depth of the fill on which the plant operated;

8 no knowledge of the degree of compaction achieved during the monitoring;

9 no control over the moisture content of the fill;

10 lack of detailed knowledge of the ground conditions beneath the fill on some sites.

These restrictions and limitations stimulated the design of an experiment in which the vibration arising from compaction plant could be studied under controlled conditions, by construction of a pilot scale earthwork. Within this experiment, the ground conditions would be constant, enabling effects attributable to the plant and fill only to be assessed. Furthermore, this approach allowed more rigorous investigations to be undertaken The design and construction of this structure and details of the testing undertaken are described in Appendix A.

Table 3 Summary of sites on which vibration data from compaction plant were acquired

Site	Plant	Fill type[*]	Approximate fill thickness (m)	Topography	Range of distances (m)
A1, Macmerry A	Bomag BW135AD	1	0.5	At grade/flat	3.0-108.2
A1, Macmerry B	Bomag BW161AD	1	0.5	At grade/flat	5.5-110.7
A1, Gladsmuir	Bomag BW6	1	3.2	Embankment/sidelong	6.6-119.0
A249, Bobbing	Stothert & Pitt T182	1	2	Embankment/flat	6.8-108.1
A47, Terrington	Bomag BW 212D	1	2	Embankment/flat	6.6-137.7
A428, Bedford	Bomag BW6	1	1	Embankment/flat	12.0-144.0
A11, Wymondham	Dynapac CA511	2	4.7	Embankment/flat	13.9-113.9
A470, Merthyr	Stothert & Pitt 72T	1	0.5	At grade/flat	2.3-115.5
A470, Dan-y-Darren	Stothert & Pitt 72T	1	1-3	Steep valley side	2.3-75.1
A5, Glyn Bends	Bomag BW10	1C	12	Steep valley side	51.0-154.0
A50, Stoke A	Bomag BW120AD	6P	4	Retaining wall backfill	17.8-88.8
A50, Stoke B	Bomag BW212D	2	2	Retaining wall backfill	14.0-86.6
M25, Pendel Wood	Bomag BW161AD	1C	1.9	Embankment/flat	6.7-109.5
A465, Resolven	Bomag BW213D	1	5	Flat valley floor	15.1-117.7
M65, Cuerden	Bomag BW6	1	2.5	Embankment	13.2-154.0
M77, Glasgow	Bomag BW213D	0[†] / 1	0[†] / 0.2	At grade/flat	1.5-110.9
M66, Rochdale	CASE W1102NCE	1	0.5	Cutting/flat	12.2-114.1
M65, Blackburn	Bomag BW6	1	1.5	Embankment/flat	3.5-72.2

[*] *Fill types are those given in the Specification for Highway Works (MCHW1): 1: granular; 1C: coarse granular; 2: cohesive; 6P: granular fill to structures.*

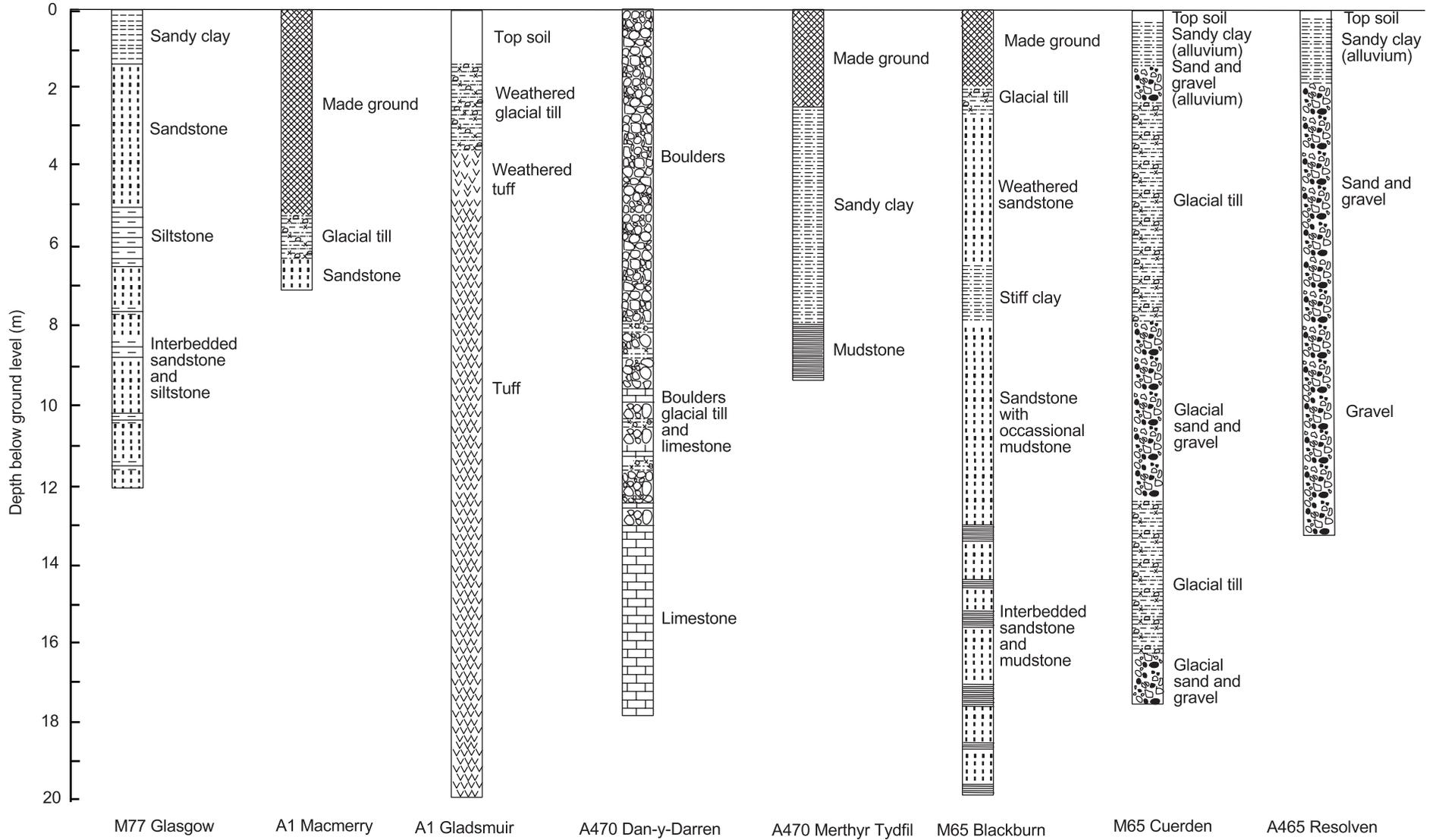[†] *Operation on natural ground/operation on fill.*

Figure 9a Simplified geological profiles for eight of the sixteen sites on which data from vibrating rollers were acquired
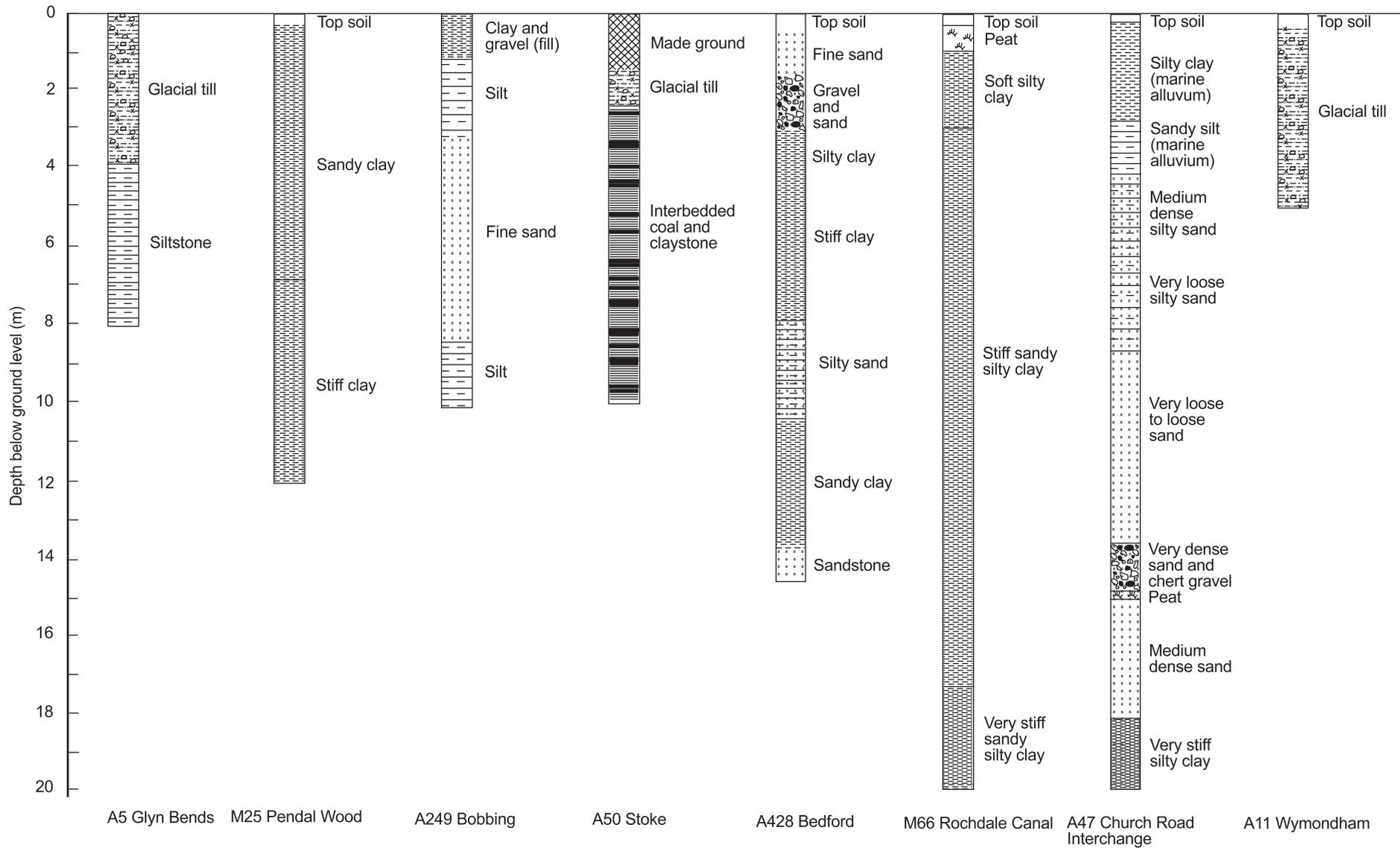
Figure 9b Simplified geological profiles for the remaining eight sites on which data from vibrating rollers were acquired

As a result of the requirements of a separate research project, an opportunity arose to acquire data from a roller operating on a variety of different fill materials within close proximity to one another, which would supplement the data from the main trial. This aspect of the data acquisition is also described in Appendix A.

4.3 Interpretation of the data from the controlled experiment

The controlled experiment had enabled more rigorous investigation of the factors which influence the levels of vibration to be investigated than had been possible on the live construction sites. Furthermore, the number of variables was less because the ground conditions and fill were constant. Consequently, the data from the controlled experiment were used to develop the prediction equation, which was subsequently verified by comparison with the data from the live construction sites. The approach which was taken to development of a predictor was to determine separately the vibration level at a common small distance from the roller, called the *source term* (v_0), and the attenuation effects (Figure 10).

4.3.1 Determination of source terms

The vibration arising from vibratory compaction close to the roller is dependent upon both the plant and the fill which is being compacted. Section 4.3.1.1 considers the effect on the level of groundborne vibration of those parameters related to the plant which are specified by the plant manufacturers. Section 4.3.1.2 describes the effect of the fill on the level of groundborne vibration.

4.3.1.1 Characteristics of the plant

A number of parameters relating to the overall size and weight of compaction plant and the characteristics of the vibratory system are specified by the manufacturers. A general impression of how the source term was related to

these parameters was achieved by normalising the data to each parameter, by dividing the distance by each of these parameters. The vibration data were then plotted against each of the normalised distance terms. While this process had the effect of reducing the scatter of the data using each parameter, normalisation by the centrifugal force caused a reversal of the trend of large plant generating the larger vibrations *ie.* the larger rollers gave rise to lower vibration levels at any distance, scaled by the centrifugal force, than did the smaller plant. Normalisation by the nominal amplitude proved to be the most successful (Figure 11), but the degree of scatter stimulated further investigation to attempt to establish a better correlated predictor by considering the source terms for each of the plant.

Ideally, the source terms for all combinations of roller and fill should be determined at a common distance to avoid extrapolation and for ease of comparison. For the main part of the controlled experiment (Appendix A), all measurements were made on the same side of the test structure (Figure 12). Therefore, the range of measurement distances for the plant operated on the hoggin was always greater than when compacting the clay. The distances to the geophone arrays for each item of plant also varied slightly because the line of trafficking was always centred on each test bed. The different drum widths and small variations in the line taken by the roller during trafficking resulted in a variation in the distances of one metre across the range of plant tested for each fill. The attenuation curve determined from measurements made at 15 different distances from a Bomag BW161AD (see Section 4.3.3) indicated that the attenuation of vibration between distances of approximately 1m and 3m from a roller is small, so the errors introduced by this variation of distance should not be significant close to the test. Furthermore, at larger distances, the variation of distance would represent a small proportion of the total distance.

The resultant peak particle velocity (ppv) at any distance caused by the operation of any roller was found to be, in
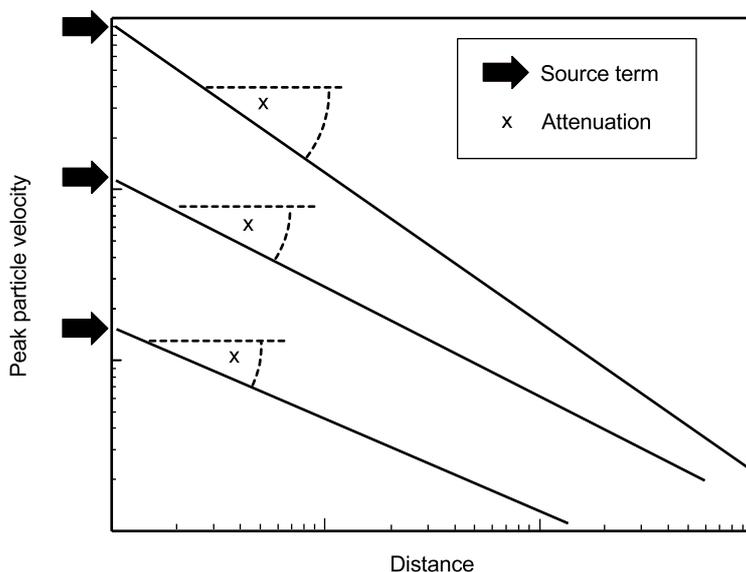
Figure 10 Illustration of the terms 'source term' and 'attenuation'

a) ppv v distance for all normal passes of plant on roller trial

b) ppv v distance/amplitude for all normal passes of plant on roller trial
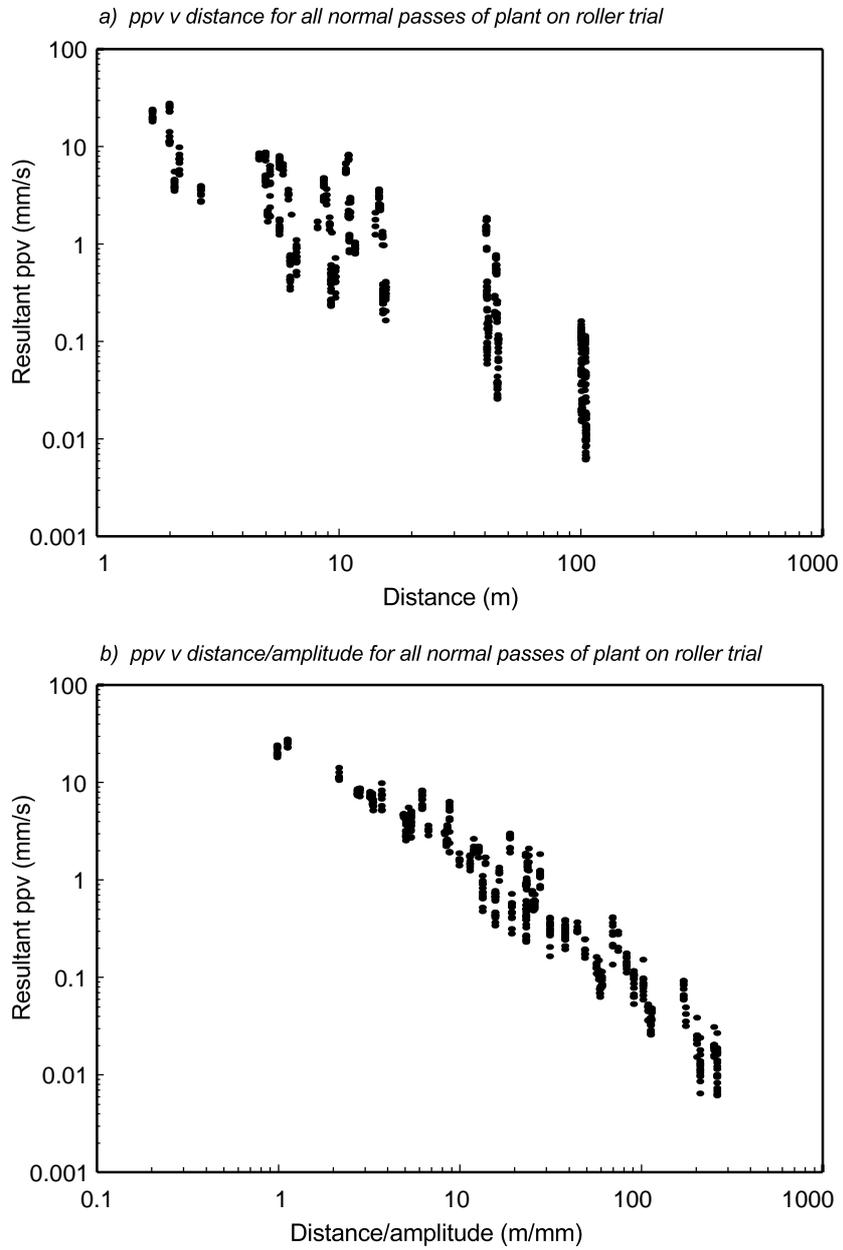
Figure 11 The effect of normalising vibration data from the roller trial by the nominal drum amplitude
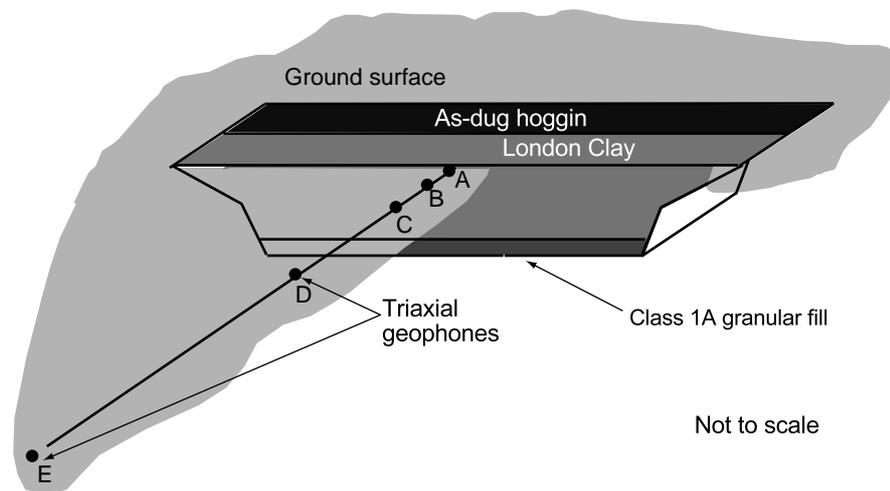

Figure 12 Schematic arrangement of test structure

most cases, higher when the roller was on the clay than when the same roller was operated on the hoggin. (This is discussed in Section 4.3.2.) Therefore, in order to minimise the number of variables, the determination of source terms for the various plant tested commenced by consideration of data from Geophone location *A* (Figure 12) for rollers operating on the clay only. This was intended to eliminate any effects arising from different interaction of the plant with the different fills; to eliminate any possible effects caused by different vibration transmission paths; and to reduce the range of distances over which the source terms were determined.

The source term was also affected by the amount of compaction that the fill had undergone, *ie* the number of passes of the roller which had been undertaken. The data from the buried uniaxial geophones showed that, for the first few passes, the number differing for each roller, the peak particle velocity increased as the number of passes increased. Data were therefore only considered for passes following the initial rapid rise shown by the plot of ppv against number of passes (Figure 13). Furthermore, for this stage of the analysis, only *normal* passes were considered, ie those where the roller was operated at a constant velocity and with the vibrator operating continuously for the entire length of the test pad. Data from the rollers operating at atypical speeds, for starting and stopping, and while changing direction were considered separately.
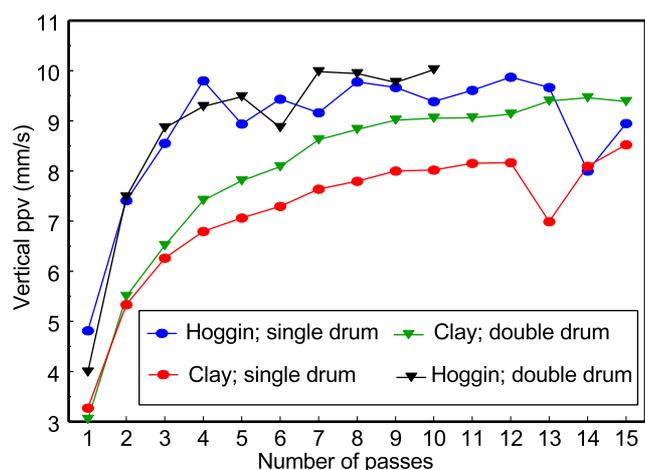

Figure 13 Increase in ppv recorded by the vertical unixaxial geophones buried beneath the test structure with increasing number of passes of the Bomag BW161AD-CV Variomatic roller

As far as was practicable, the precautions described above restricted the factors affecting the source term vibration level to parameters which were dependent only on the rollers.

To ensure that the prediction method would be of practical use, it was necessary to make use of readily available information, such as the specification data provided by the plant manufacturers. Parameters relating to the drum and vibration system which are typically specified are the static linear load, the drum width, the

frequency, the nominal amplitude and the centrifugal force. To investigate how the vibration level was influenced by each parameter it was necessary to, as far as possible, isolate each of the variables. The investigation of the effect of these parameters on the resulting vibration level is described below.

Centrifugal force and frequency
For most vibrating rollers currently in use, vibration is generated by rotation of an eccentrically loaded shaft mounted axially within the vibrating drum. The dynamic force arising from this vibration is described in manufacturers' data sheets as the centrifugal force. For consistency with the manufacturers' descriptions, the term centrifugal force will be retained herein, although it is acknowledged that the term is not strictly correct.

Although the plant manufacturers specify the operating frequency and centrifugal force, spectral analysis revealed that, for many of the rollers tested, the actual frequency of vibration differed from that quoted. The centrifugal force is a function of the eccentric mass, the distance of the centre of gravity of the eccentric from the axis of rotation and the frequency of rotation. Since the first two of these parameters are fixed, if the frequency changes then the centrifugal force must also change. The actual centrifugal force was therefore calculated by determining the operating frequency by spectral analysis of the vibration data.

The operating frequency for all rollers, except for the particular Ingersoll-Rand DD65 tested, was found to be reasonably consistent where the rollers were operated normally, although the actual frequency commonly differed from that specified by the manufacturers. The Ingersoll-Rand DD65 was found to have operated on the clay at frequencies ranging from 37Hz to 53Hz. Data were recorded from this roller operating in single vibrating drum mode and double drum mode. For both cases, the resultant ppv at the closest monitoring location decreased as the frequency, and therefore the centrifugal force, increased (Figure 14a).

It has been shown by Yoo and Selig (1979) that the ratio of the generated dynamic force to the transmitted force is not constant with frequency (Figure 15). The force transmitted to the fill increases much more slowly than does the generated force over the normal range of operating frequencies of vibrating rollers. Similarly, it has been found that the amount of compaction is not directly related to the generated dynamic force (Parsons, 1992). Therefore, this parameter is unlikely to be useful for development of a ground vibration predictor. Furthermore the preliminary analysis described above had indicated that the centrifugal force is not an appropriate parameter for predicting vibration.

The Ingersoll-Rand SD150 single drum roller allowed the vibrator to be operated at a range of frequencies from zero to 27Hz. The data showed a distinct increase in resultant ppv with increasing operating frequency (Figure 14b). Vibrating rollers are designed to operate at a frequency above the resonant frequency. The model proposed by Yoo and Selig (1979) indicated that below this frequency the force transmitted to the fill increased with increasing frequency, whereas in the frequency range
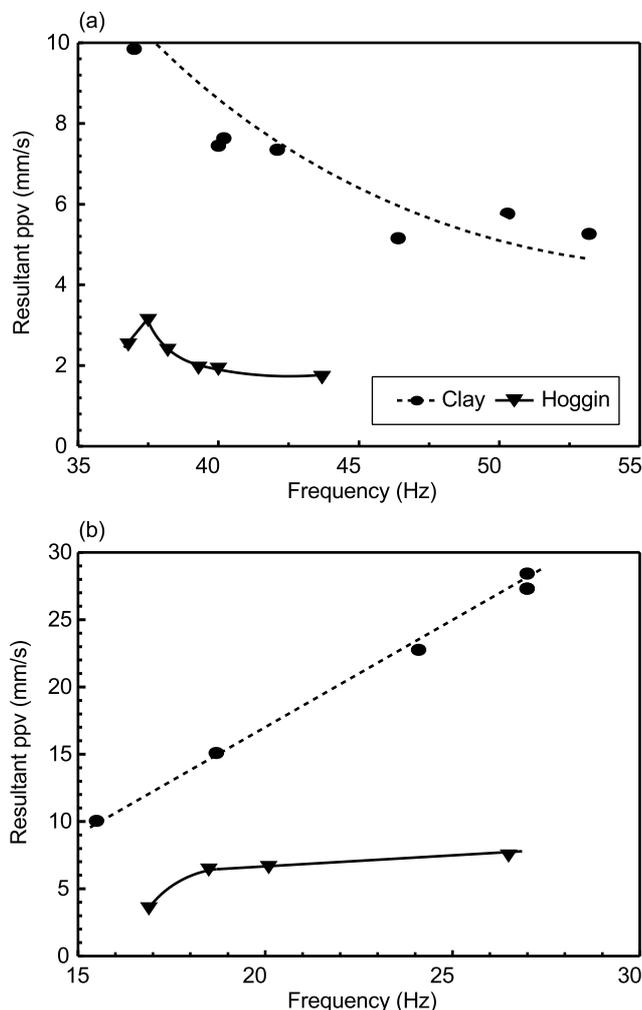
Figure 14 Effect of frequency on ppv for two vibrating rollers: (a) the Ingersoll-Rand DD65 (b) the Ingersoll-Rand DS150

within which rollers are designed to operate, the transmitted force decreased slightly with increasing frequency. The resonant frequencies were determined to be approximately 29Hz for the DD65 and 22Hz for the SD150. The response therefore fitted the trend modelled by Yoo and Selig, except that the vibration from the SD150 continued to increase with increasing frequency above the resonance frequency.

The two Ingersoll-Rand rollers were the only plant tested which operated over a significant range of frequencies. While the SD150 exhibited a marked increase in vibration level over the range of frequencies at which it was tested, the decrease in ppv which occurred for the DD65 was considerably less. Since rollers are designed to operate at frequencies above the resonance, the frequency *per se* appears to be unlikely to make a significant contribution to the resulting level of vibration. However, if the operating frequency were to coincide with the characteristic frequency of the soil (Table 4), then problems might be exacerbated. Consideration should be given to the preferred frequency of the ground and the operating frequency of the roller on sensitive sites.
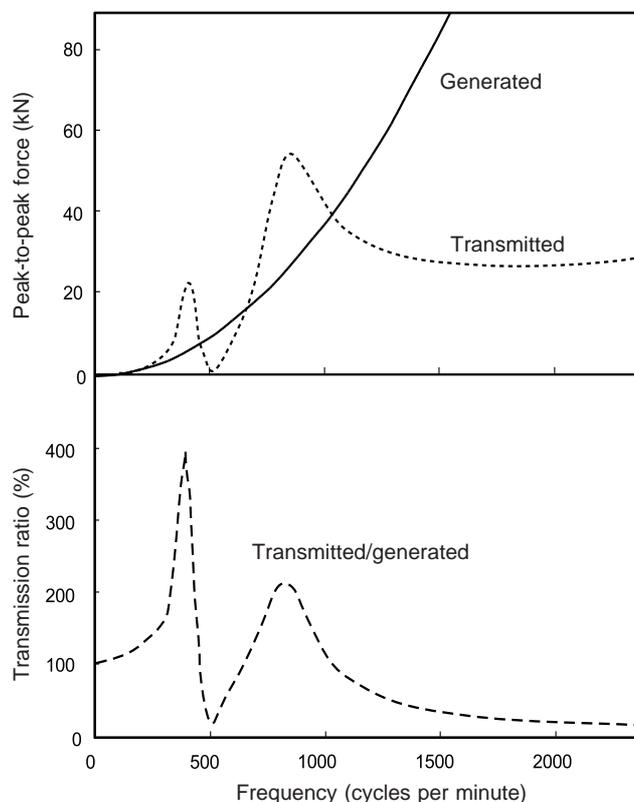
Figure 15 Variation of the ratio of transmitted to generated force with frequency (after Yoo & Selig, 1979)

Table 4 Characteristic frequencies for soils and rocks observed during piling coperations (Head and Jardine, 1992)

Material	Frequency (Hz)
Very soft silts and clays	5 - 20
Soft clays and loose sands	10 - 25
Compact sands and gravels and stiff clays	15 - 40
Weak rocks	30 - 80
Strong rocks	>50

Static linear load

Lewis (1961) found that the static weight per unit width of the drum gave a 'reasonable guide' to the likely performance of vibrating rollers. The importance of this parameter for determining the ability of a roller to compact fill is reflected by its use to categorise vibrating rollers in the Specification for Highway Works: it is on this parameter alone that the compaction requirements for each combination of plant and fill type are specified.

Despite the importance of this parameter for compaction, it was not possible within the trials to undertake tests which isolated the static linear load as the only variable whilst maintaining all other parameters constant. However, three tandem rollers which operated at broadly similar amplitudes, frequencies and travel speeds, enabled an indication of the effect of static linear load on the resulting vibration level to be obtained. These rollers were the Bomag BW135, Benford TV75 and Ingersoll-Rand DD65.

Although the data showed a considerable degree of scatter (Figure 16), an upper bound envelope to the data

PPV at Geophone A
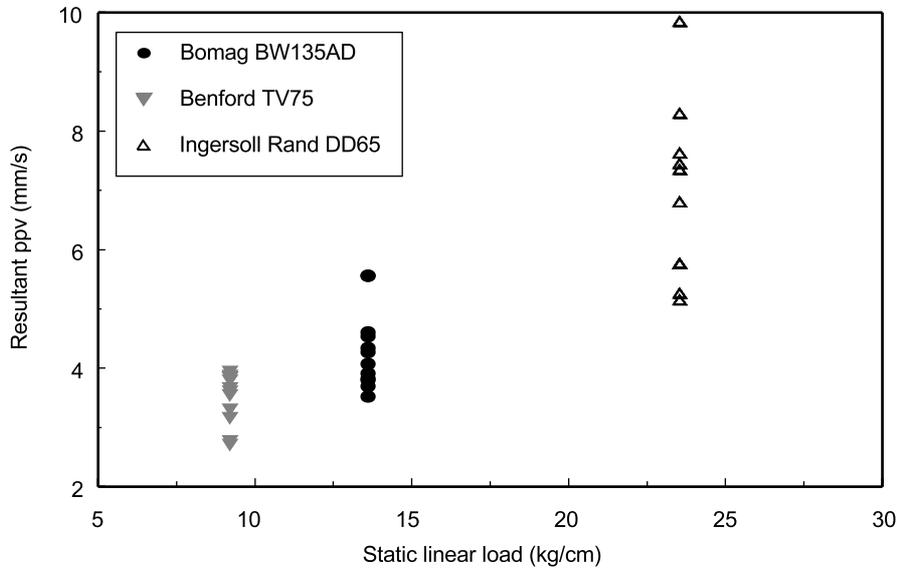
Figure 16 Effect of static linear load on ppv for three tandem rollers

suggested a linear relation between the static linear load and the resultant peak particle velocity. The preliminary analysis which normalised the distance data according to the static linear load had also indicated a strong correlation with this parameter.

Nominal amplitude

Several of the rollers tested during the controlled experiment had the option to operate at either a low or high amplitude setting. Most of the experimental work was undertaken at the high amplitude setting, since it was anticipated that this would be the worst case, but some additional testing was also undertaken at the lower amplitude on the fully compacted fill. Although for each item of plant there was clearly an increase in the resultant ppv with increasing amplitude, it was only possible to acquire data for two amplitude settings for each roller, so direct determination of a relation between the nominal amplitude and the peak particle velocity was not possible.

Normalisation of the whole data set for the controlled experiment according to each of the individual vibratory system parameters, as described above (Section 4.3.1), yielded the highest correlation when the nominal amplitude was used (Figure 11).

Travel speed

The travel speed affects the amount of compaction achieved to the extent that in order to comply with the Specification for Highway Works, a travel speed of between 1.5 and 2.5km/h is required for most types of vibrating roller. If a higher speed is used it is necessary to increase the number of passes in proportion to the increase in speed.

The ground vibration arising from a vibrating roller was also found to be dependent upon the travel speed. From tests carried out using a Bomag BW161AD over a range of speeds between zero and 6.7km/h, it was found that the

resultant ppv was approximately related to the inverse of the square root of the travel speed (Figure 17). A similar relation was observed from the BW161AD-CV *Variomatic*, although fewer data were available. Therefore, increasing the roller speed will reduce the vibration, but a greater number of passes will be required which will increase the duration of the disturbance.

$$ppv \ \alpha \ \frac{1}{\sqrt{speed}}$$
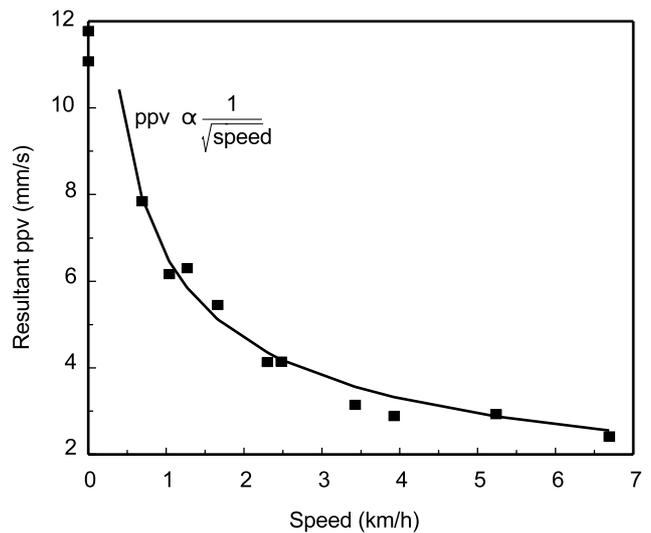
Figure 17 Effect of travel speed on the ppv recorded at a distance of 2.1m during operation of the Bomag BW161AD roller

Number of drums

Operation of a roller with two vibrating drums should transmit twice the energy into the fill as would a single drum roller having the same vibrator specification. It would therefore be expected that the particle velocity arising from a double drum roller would be $\sqrt{2}$ (=1.41) times that from the single drum roller.

During the experiment, some passes of the tandem rollers were undertaken while only one drum was vibrating. The data from passes with only one roller vibrating were compared with passes immediately preceding for which both drums were vibrating (Figure 18a). The mean value of the ratio of two-drum vibration to that arising from a single drum was 1.424 (with a standard deviation of 0.315), which is very close to the theoretical value of 2. This value was derived from the Bomag BW135AD, Benford TV75 and Ingersoll-Rand DD65 rollers.

In addition, the Ingersoll-Rand DD65 had been operated on the clay for many passes with only one drum vibrating. Data from the two sets of tests were compared (Figure 18b) and least squares regression lines fitted to the log-transformed data. This revealed a ratio of 1.46 between the resultant ppv arising from double and single drum operation, which is again very close to √2. The ratio remained constant over the range of distances from 2m to 101m.

4.3.1.2 Consideration of energy transmitted into the fill
The preceding section defined the relations between the ppv arising from vibrating rollers and the roller parameters. While this approach proved to be reasonably successful, the problem was also approached theoretically, on the basis that the vibration level should be related to the energy transmitted into the fill. Research undertaken by Yoo and Selig (1979) showed that the transmitted dynamic force was not simply related to the generated dynamic force, so approximation of the energy by this route was not appropriate.

An alternative approach was based on the assumption that, during each cycle, the vibrating drum was raised through a distance equal to the nominal drum amplitude (A) and dropped to the ground. The energy input could then be approximated to the potential energy gained and dissipated during this cycle. The weight, which includes the weight of the drum and a contribution from the whole of the plant, was calculated from the static linear load (L_s) and the drum width (w). The ppv would then be expected to be proportional to the square root of this energy term, ε_p. The value of ε_p was calculated from:

$$\varepsilon_p = AL_s wg \tag{6}$$

where g is the acceleration due to gravity.

Figure 18 Effect of the number of vibrating drums on the level of groundborne vibration

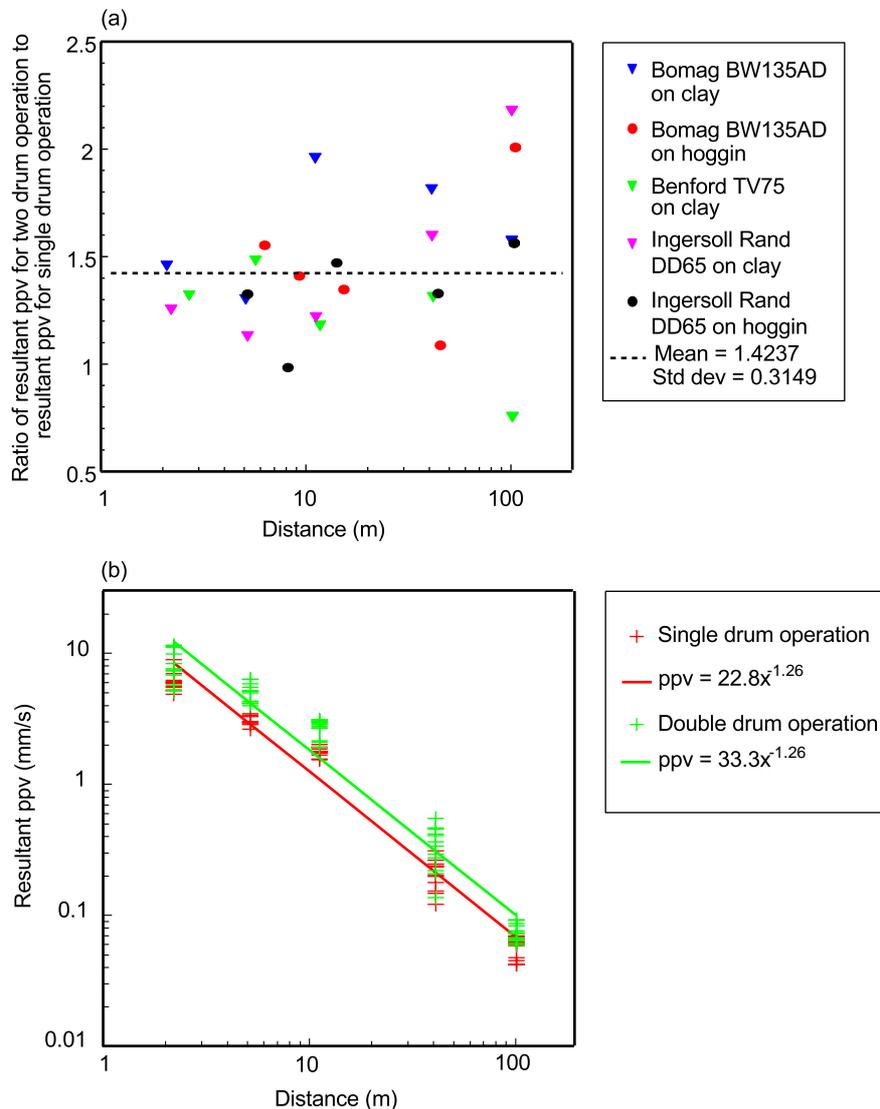To eliminate effects due to data from different distances and fill types, the analysis again focused on the data from the geophone closest to the test area acquired while the rollers were operating on the clay. The resultant ppv (v_0), was plotted against the square root of ε_p (Figure 19). A linear regression analysis yielded a relation of;

$$v_0 = 2.07\sqrt{\varepsilon_p} - 1.40 \qquad (7)$$

Where v_0 is in mm/s and A, L_s, w and g are in SI units, giving a value of e_p in joules. This relation did not account for the travel speed (km/h) or the number of vibrating drums, but provided a means of determining the resultant ppv at a distance of 2m from a roller operating on the clay. The next step was to investigate the effect of the fill type on the vibration level.

4.3.2 The effect of fill properties on the vibration level

During the controlled trial, it was observed that the vibration level was affected by changes in the properties of the fill as compaction progressed. The first pass of any item of plant always gave rise to lower levels of vibration than subsequent passes. For some of the rollers, a number of passes were required before no further increase in vibration level occurred. The increase in the dry density of the fill, measured using a nuclear density gauge, reflected the changing level of vibration (Figure 20). This might be expected since the ground vibration arises from energy transmitted to the fill which is not used in compacting the fill. At refusal, therefore, the vibration level might be expected to remain approximately constant for a particular combination of plant, operation and travel speed.

In practice, changes in vibration levels will arise through local variations in the fill and differences in the amount of trafficking the fill has received. It was noticeable during the controlled trial that the vibration was reduced if the roller went only slightly off the line of the strip of fill which had been previously compacted.

There was also a difference in the vibration level recorded from plant operating on the two different fill types used for the controlled experiment. Vibration from rollers operating on the clay was greater than when operating on the hoggin in all cases except for the Bomag *Variomatic*. One possible explanation is that some interaction between the roller and the fill, or some characteristic of the fill, effectively reduced the source vibration level such that the energy transmitted into the environment was lower for operations on the hoggin than on clay. Alternatively, the same amount of energy may have been radiated from the roller operated in both situations, but the energy transmitted into the hoggin was attenuated more because an additional acoustic impedance mismatch was encountered, which would reduce the energy reaching the geophones, since the clay was located between the hoggin and the geophones for all the measurements (Figure 12).

In order to investigate whether the acoustic impedance interface between the clay and the hoggin affected the level of vibration, measurements were made simultaneously on both sides of the test area while a roller was operated on the hoggin. The vibration levels recorded on both sides were the same, at any distance, which demonstrated that the additional interface between the two fill materials had no significant effect on the vibration. The effect must therefore have arisen through the interaction between the roller and the fill.

Data from the vertical-axis geophones buried beneath the test area showed that the vibrations recorded at the
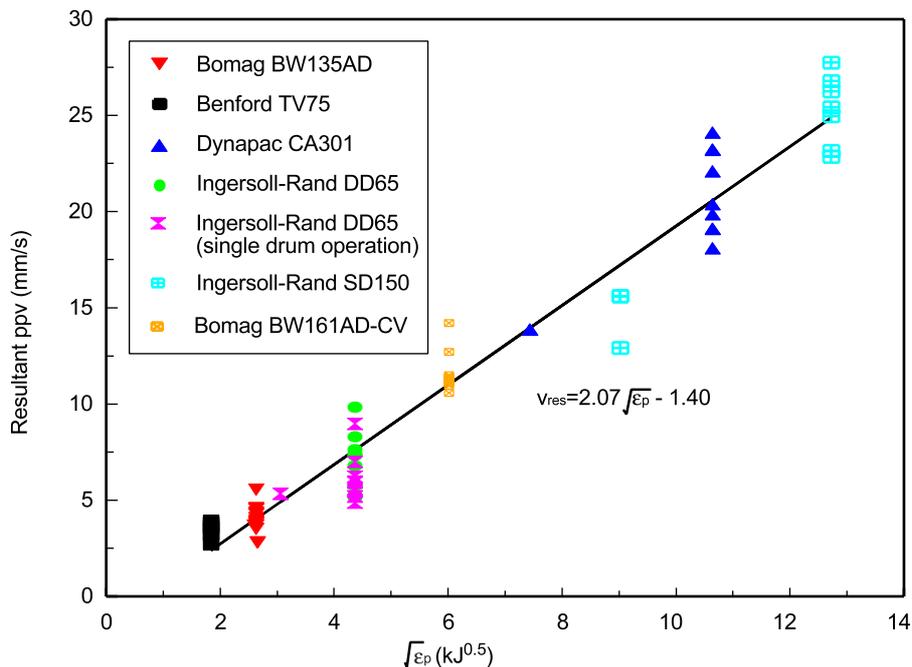
Figure 19 Relation between the ppv recorded at Geophone A during the roller trial and a function of nominal amplitude, static linear load and drum width
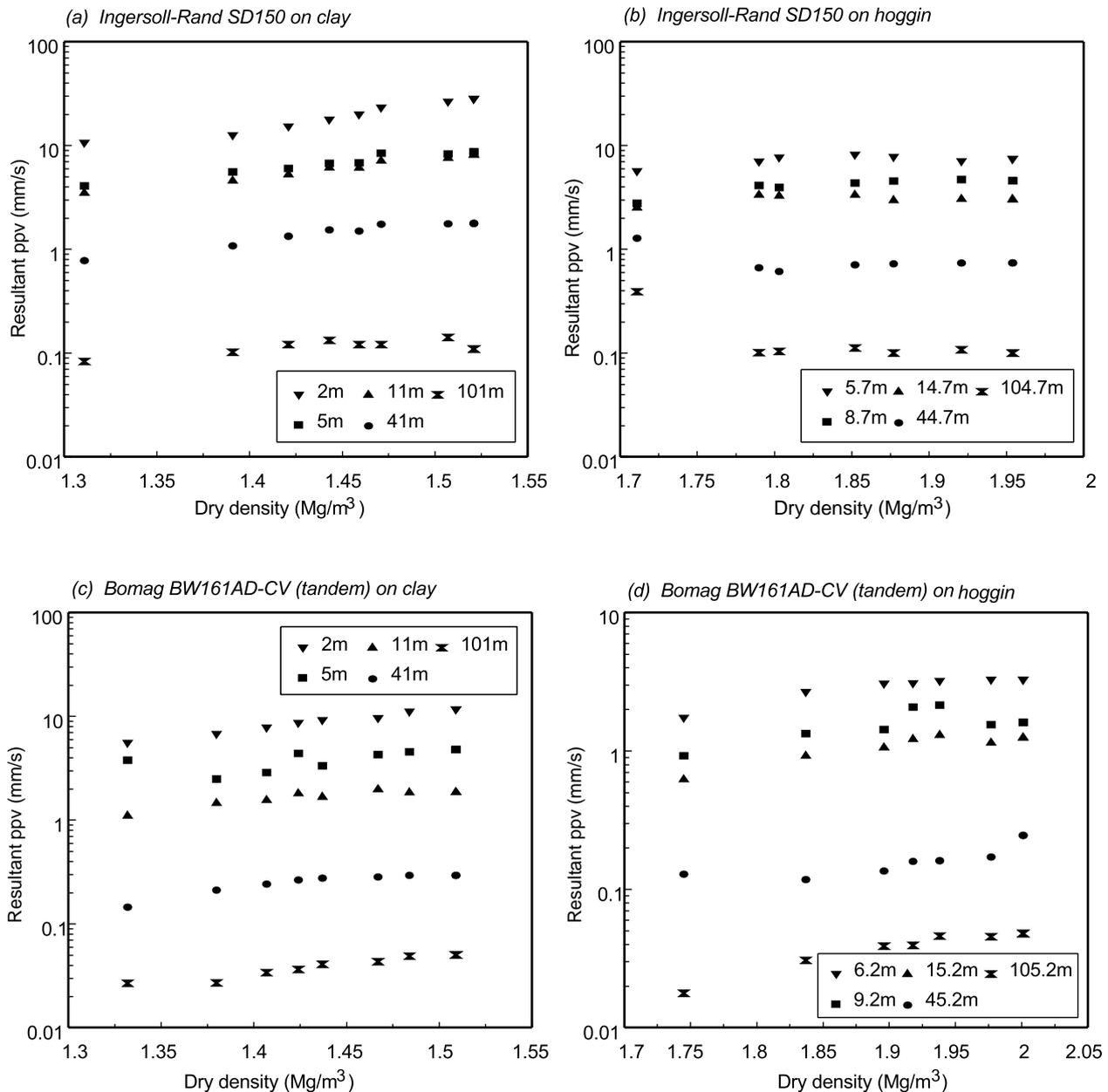
Figure 20 The effect of dry density of each fill material on thepeak particle velocity recorded at each geophone. (a) and (b) are data from a single drum vibrating roller, (c) and (d) are data taken from a tandem vibrating roller

base of the clay were greater than those beneath the hoggin in most cases. This provided further evidence that the difference in vibration level arose through different behaviour of the roller on the two fill materials. Measurements made with accelerometers secured to the vibrating drum of four of the rollers also showed that the velocity of vibration of the drum was consistently higher while operating on the clay than on the hoggin.

Dynamic stiffness measurements were undertaken in November 1997 using continuous surface wave tests (Matthews *et al*, 1996). The fill had been exposed to the weather for more than 12 months following completion of the main phase of the trial, so the surface had become soft, particularly on the clay bed. However, the relative stiffnesses of the materials were assumed to have been similar to that during the tests. The stiffness of the hoggin was found to be approximately an order of magnitude

higher than the stiffness of the clay. Therefore, the vibration generated may be a function of the stiffness of the fill, with larger vibration arising from plant operating on less stiff fill. A similar trend has been observed by Watts (1992) for vibration arising from road traffic and by Hiller (1991) for railways.

Some of the data from the rollers, however, provided contradictory evidence. The Bomag BW135AD was tested on the hoggin initially while the fill was wet of its optimum moisture content and again when it had been allowed to dry for several days. The vibration level was found to be lower for compaction of the wet fill than when it was at a lower moisture content, and was therefore stiffer.The vibration level recorded when the BW135AD was operated on the clay was, however, greater than both of the sets of data from the hoggin, as was the case for most of the other plant tested. Tiedemann (1970) also reported higher levels of

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vibration from compaction of fill which was dry of optimum than those recorded when the same material was compacted at a moisture content above optimum.

The Bomag *Variomatic* also behaved differently to the other rollers tested, giving rise to higher vibration when operated on the hoggin than when on the clay. This may be a result of the effect of the different vibratory mechanism in this roller which generates a vertically polarised vibration when operated on soft ground. Conventional vibrating rollers generate vibration by rotation of a single eccentrically weighted shaft, which gives rise to an elliptical drum motion. The *Variomatic* uses two contra rotating eccentrically loaded shafts which enables the direction of the vibrating force to be varied (Byles, 1997).

As part of a separate research project, a 36m length of road, founded on contiguous sections of subgrades of chalk, London clay and a silty sand was constructed (Appendix A). Each of the subgrade materials were placed in three separate bays (Figure 21), the material in each bay being at a different moisture content. The stiffness of each

subgrade was determined using a variety of non destructive methods (Evans, 1998). This trial road presented an opportunity to measure the vibration levels arising from a constant source of vibration operating on a number of different materials. The measurements were undertaken following the placement and compaction of the capping material. Trafficking was only possible on seven of the nine bays since two of the sand bays were too weak to support the roller with the vibrator in operation.

Comparing the vibration levels with the stiffness for all materials showed no correlation between the stiffness and the peak particle velocity. For individual materials, however, there was some indication that the particle velocity was higher when the stiffness was higher for a particular material (Figure 22). Yoo and Selig (1979) found that the force transmitted into the fill increased as the fill stiffness increased, which may account for higher vibration arising from stiffer materials. These results, however, contradict the findings of the main part of the research for which the vibration was generally higher
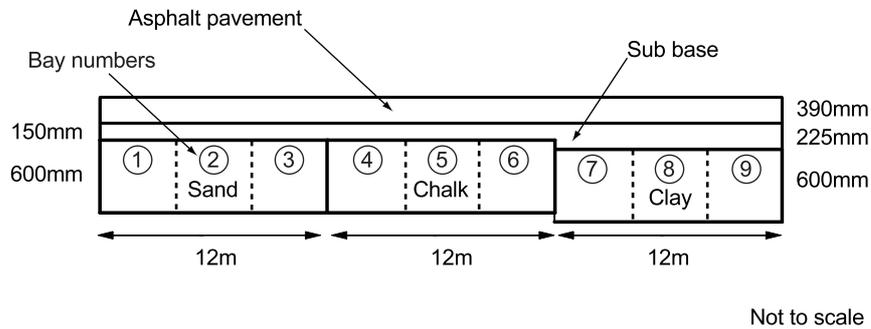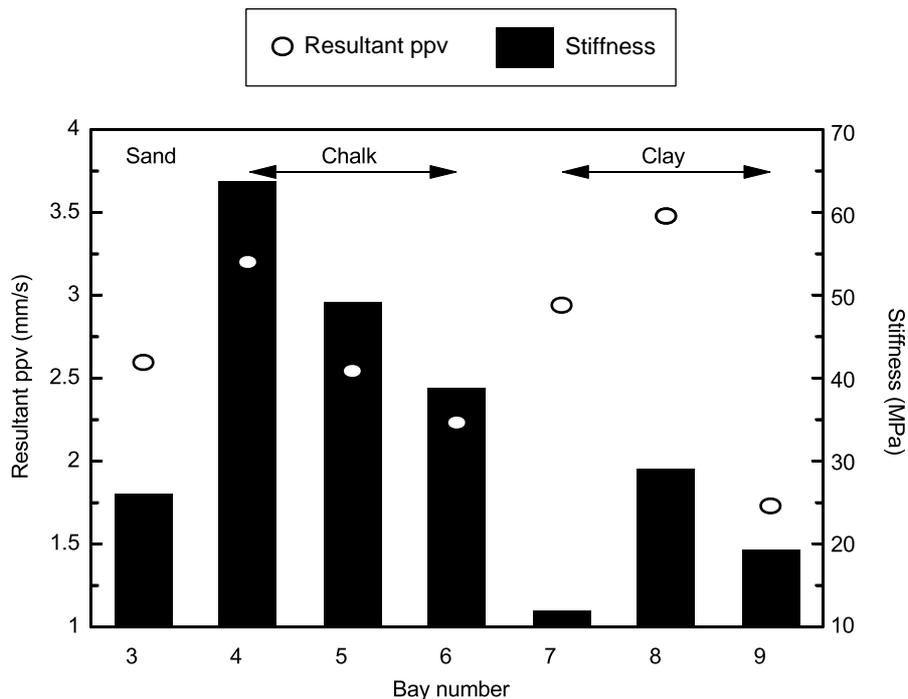

Figure 21 Section through trial road

Section 22 Relation between the trial road subgrade stiffness and the ppv recorded at a distance of approximately 4m

during compaction of the clay than it was for the hoggin, which had a higher stiffness. It was therefore concluded that parameters other than the stiffness have an effect on the generation of vibration.

In summary, the vibration level arising from vibratory compaction was found to be influenced by the properties of the fill. The vibration was seen to increase as the dry density increased during compaction and become approximately constant as the state of compaction approached refusal. For most plant, compaction of the clay gave rise to higher levels of vibration than did compaction of the hoggin. There was a general increase in vibration level with increasing stiffness *for a given material*, but there appears to be an influence from some other material property, since the trend was not apparent when comparing different materials. Given the scatter in the data and the inability to attain a qualitative conclusion, the fill type has been excluded from the prediction methodology.

4.3.3 Attenuation of vibration

The conventional method of presentation of ground vibration data is a straight line on a log-log plot (for example New, 1986). A number of authors have suggested that the data field is actually curved (for example Richart *et al*, 1970), which causes extrapolation of straight line relations to over estimate vibration levels at the extremes of the data (for example Gutowski *et al*, 1977). Dowding (1996) pointed out that use of a power law predictor benefits from providing a conservative approach. Using a non-linear regression relation, to fit a curve to field data, may provide a better representation of the data, particularly if the data were acquired over a wide range of distances. Although a curved relation will limit over-estimation caused by extrapolation, prediction of vibration outside the data field should still be avoided. In any assessment, measurements should be made over a range of distances which extends to or beyond both limits of distances over which vibration is to be predicted.

Attenuation of groundborne vibration as energy propagates through the ground occurs through geometric effects and, to some extent, by frictional damping. Although the significance of the contribution of the latter is unclear for vibration from civil engineering sources (Section 1.3.1), the rate of attenuation of vibration in the ground is greater than would be expected from simple geometric attenuation in a homogeneous half space. Additionally, in many cases, the field data exhibit a curvature when plotted in log-log space, from which it may be inferred that there is a contribution from mechanisms other than just geometric spreading. The following section discusses the interpretation of the attenuation of vibration data from the controlled trial.

The nature of the attenuation of vibration from each piece of plant was determined from least squares regression lines fitted to the resultant ppv plotted against distance in log-log space, although in many cases the data field exhibited a degree of curvature. The gradients of these regression lines showed a poorly correlated increase in the rate of attenuation with increasing operating frequency of the plant (Figure 23).
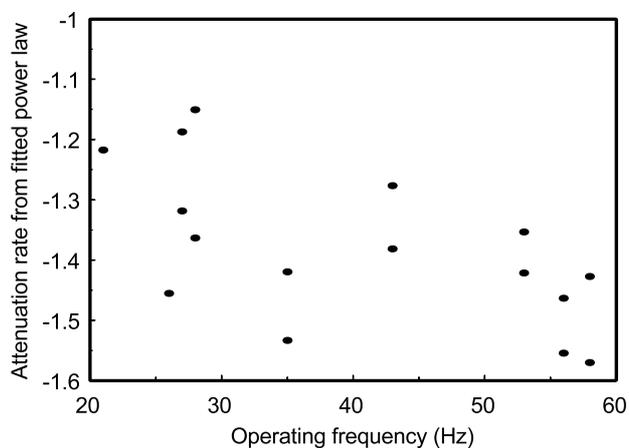
Figure 23 The relation between the operating frequency and the rate of attenuation of vibration for compaction plant operated during the roller trial

Fitting a curved relation of the form of Equation (5), by a process which had iterated the parameters β and α the ppv (v_0) at a reference distance, showed no relation between and the roller operating frequency. The effect of frequency on attenuation rate was therefore investigated further by plotting the root mean square (rms) particle velocity in each one third octave frequency band against distance, for each item of plant. The attenuation rate in each frequency band was then determined by fitting a power law to the data, as illustrated by the example presented in Figure 24. The attenuation rate, defined by the exponent to the distance (b), was then plotted against the centre frequency of the $1/3$ octave frequency bands (Figure 25). The trend exhibited by the data could be represented by a relation relating b to the logarithm of the frequency (f).

By incorporating this relation between attenuation and frequency in an equation of the form proposed by Mintrop (1911) (Equation 5), an attempt was made to fit a curve to the data from the Bomag BW161AD. A well defined attenuation curve had been obtained for this piece of plant by acquiring data from 15 different horizontal distances (Figure 26). However, it was not possible to fully represent the observed attenuation using an expression of the form proposed by Mintrop. Whilst the model provided a good fit at larger distances, the behaviour at distances of approximately 1m to 3m were poorly modelled, with an over prediction arising from the curve.

Mintrop's equation describes the attenuation behaviour of surface waves radiating from a point source. While vibrating rollers may be approximated to a point source of energy at large distances, closer to the source a roller is more closely represented by a line source, since energy is transmitted into the ground along the contact between the fill and the vibrating drum. The wavefronts radiating from a roller are therefore not circular, as they would be from a point source, but take on a form which would have semicircular ends separated by linear sides of a length equal to the width of the vibrating roller. Rather than the surface wave energy spreading out over an increasingly large circumferential length, given by $2\pi r$, where r is the radial distance from the vibration source, the energy is
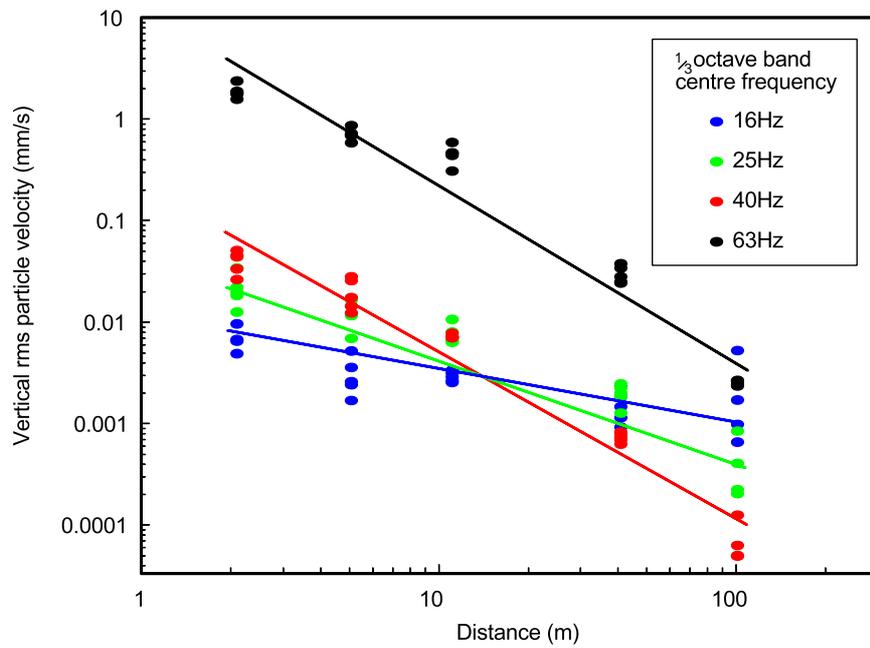
Figure 24 Variation of rate of attenuation with vibration frequency
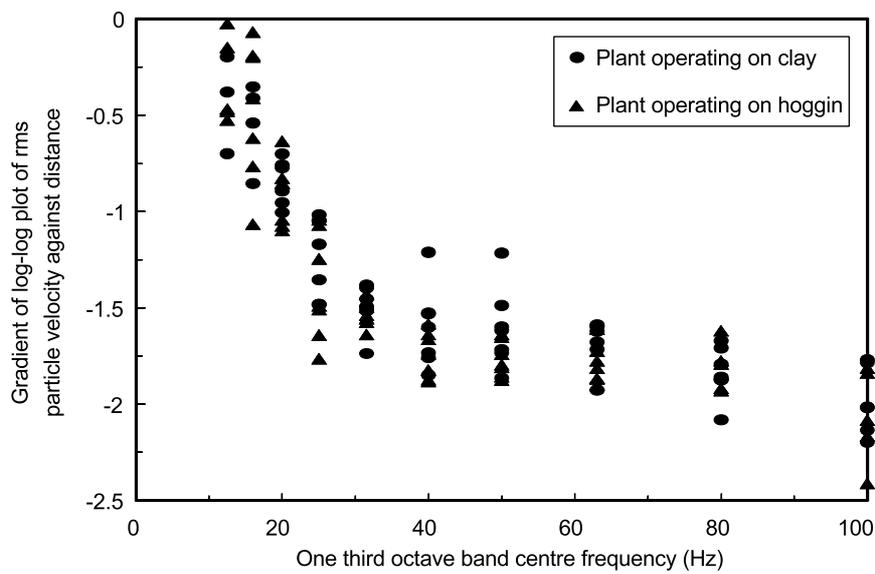
Figure 25 Attenuation vs. frequency for PV data separated into third octave bands
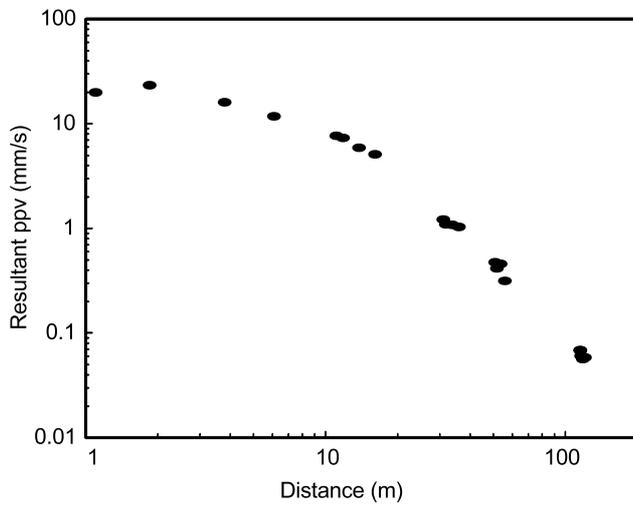
Figure 26 Attenuation curve for vibration from a Bomag BW161AD operated on natural ground

distributed over a length of $(2\pi\ r+2w)$, where w is the width of the drum. The use of a model based only on geometric spreading from a line source could not adequately represent the curvature observed in the data and an additional exponential term was required to make the gradient of the model sufficiently steep at distances in excess of approximately 70m. Equation 5 was therefore modified to:

$$v_r = v_0 \sqrt{\frac{r_0 + w}{r + w}} e^{-\alpha(r-r_0)} \tag{8}$$

The curve described by this equation has a lower gradient at close range and is therefore more representative of the data than is Mintrop's original equation (Figure 27).

As described above, the attenuation rate on the site of

the controlled trial had been found to have a frequency dependence such that it could be represented by an expression including the logarithm of the frequency. It was found that the best representation of the data was produced by an expression for α (units of m^{-1}) which included the natural logarithm of the frequency f:

$$\alpha = 0.01\ 1n(f) \tag{9}$$

Combining Equations 8 and 9 gave the following expression for the resultant ppv (v_{res}; mm/s) at a distance x (m) from the closest edge of rollers operating on clay on the controlled trial site:

$$v_{res} = v_0 \sqrt{\frac{2+w}{x+w}} e^{\left[-0.01\ 1n(f).(x-2)\right]} \tag{10}$$

where v_0 is the resultant ppv (mm/s) at the reference distance of 2m, given by Equation (7)

w is the width of the roller (m); and

f is the operating frequency of the roller (Hz).

The data from which this expression has been derived were acquired at slightly different distances from each of the rollers tested. The reference distance has been taken as 2m in the above expression. This introduces some errors, the largest of which relates to the data from the Benford TV75, which was recorded at 2.7m from the roller. While this represents a difference of 26 per cent in the distance, the resulting prediction is only affected by 10.8 per cent. Given the scatter in the data, this is considered to be an acceptable approximation.

Equation 10 has been fitted to the data from the controlled trial in Figure 28. It was found that in most cases the vibration was lower at any particular distance from rollers operating on the hoggin than when operating on the clay, but not to a consistent degree. Predictions for
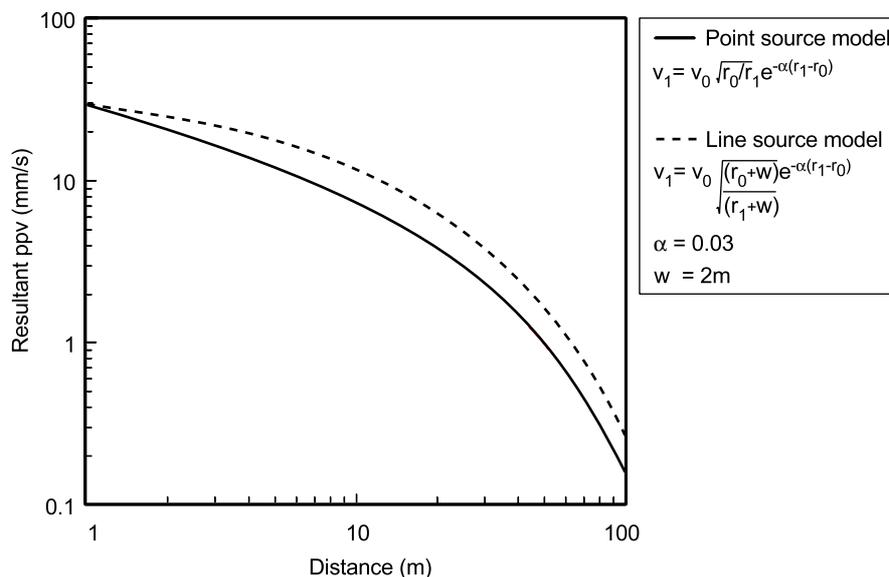
Figure 27 Attenuation described by the relation proposed by Mintrop (1911) for a point source and the effect of assuming a 2m long linear source of vibration
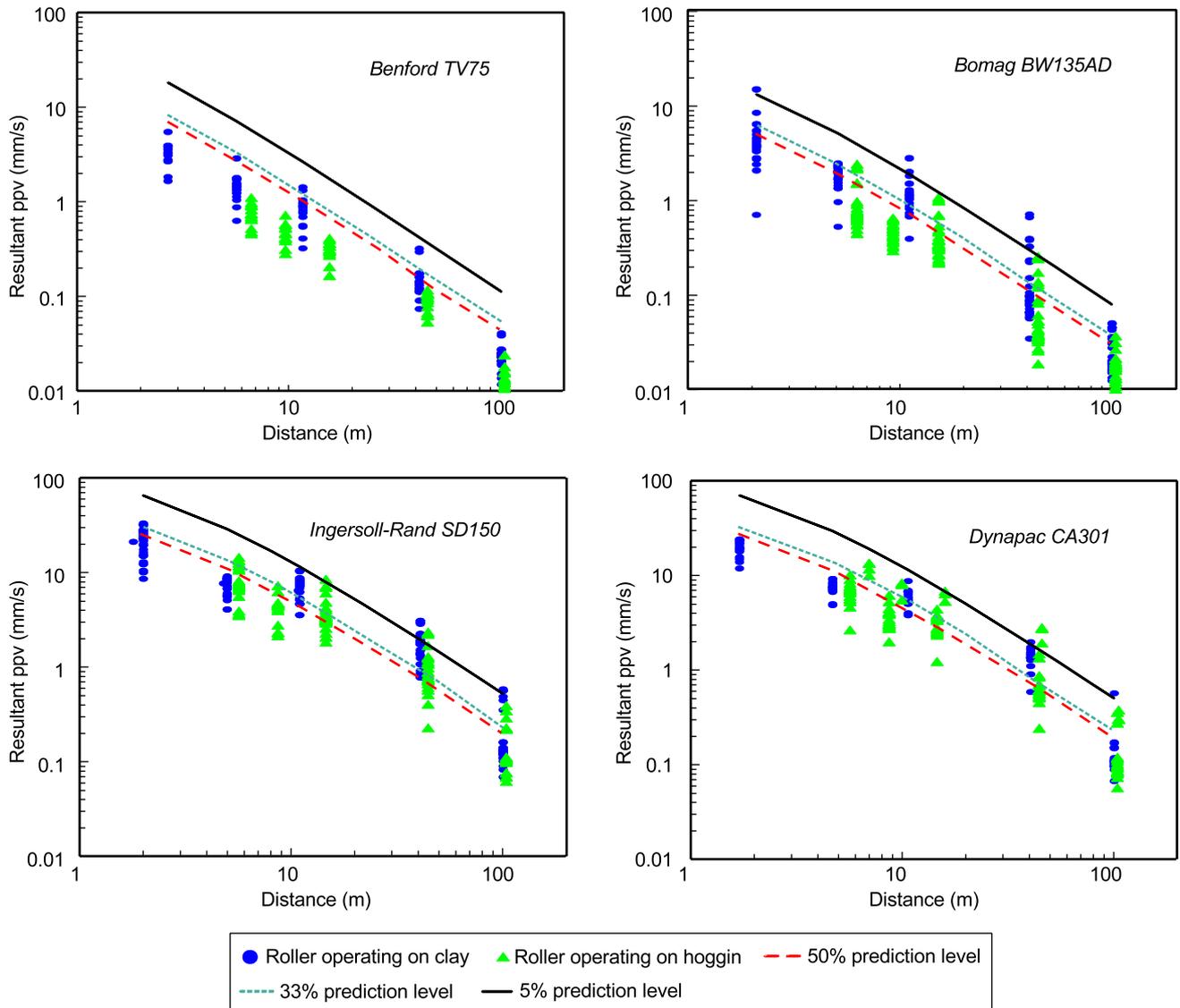
Figure 28 Equation 10 fitted to data from the controlled trial

rollers operating on hoggin may therefore be slightly conservative compared with those for rollers operating on clay. The three lines presented on each part of the figure illustrate predicted vibration levels, which would not be expected to be exceeded in 50%, 33% and 5% of cases.

4.4 Application of predictive model to the data from live construction sites

An attempt was made to determine the effect of different ground types on the attenuation of vibration from the data gathered on the live construction sites. This is a complex problem which could be affected by a large number of variables including the number, thickness, density and stiffness of each layer, the depth of the water table and the topography of the site as well as the operating frequency of the plant.

The approach taken to determining the effect of the ground on the attenuation was to examine the slope of the attenuation curve for each item of plant at each site. Regression lines were calculated for a range of third-octave

band centre frequencies from 10Hz to 100Hz in a similar manner to that described for the data from the controlled trial. Plots of attenuation slope against third octave band centre frequency showed the same general trend of an increase in attenuation with increasing frequency, as seen in the controlled trial. However there was a great deal more scatter, particularly above about 30Hz. No relation could be identified between the attenuation gradients at any frequency and the characteristics of the ground at the sites.

While the model given by Equation 10 successfully represented the data from the controlled trial, when applied to the data from live sites it did not satisfactorily predict the observed levels of vibration. Given the amount of scatter observed in the data and the limited success of the model derived above at predicting vibration levels from live sites, an alternative, simpler empirical model was sought. This would also have the advantage of being more readily applicable to assessment of potential problems on site.

It had been determined from the data from the controlled trial that, in general, the vibration levels were better correlated with the nominal amplitude than they

were with any of the other parameters of the vibratory system. In addition, it had been found that for small distances, it was important to consider the roller as a line source of vibration when modelling the attenuation. In particular, this reduced the over estimation of vibration levels at small distances. To account for this factor, the distance term used in the equation was taken to be the sum of the distance from the near edge of the roller and the width of the vibrating drum. While this modification of the distance parameter improved the fit of the model, the attenuation rate required an index of 1.5; the vibration was found to attenuate more rapidly than a direct inverse distance relation. A further consideration was that a factor of √2 existed between vibration levels arising from single drum and double drum compaction (Section 4.3.1.1).

Taking account of these points and carrying out a linear regression analysis on the data from normal compaction passes from both the controlled trial and from live sites, the following equation is proposed for the prediction of vibration from normal compaction passes:

$$v_{res} = k_s \sqrt{n} \left[\frac{A}{x+w} \right]^{1.5} \qquad (11)$$

where: k_s = 75, with a 50 per cent probability of the vibration level being exceeded;

k_s = 143, with a 33 per cent probability of the vibration level being exceeded;

k_s = 276, with a 5 per cent probability of the vibration level being exceeded;

n is the number of vibrating drums;

A is the nominal amplitude of the vibrating roller (mm);

x is the distance along the ground surface from the roller (m); and

w is the width of the vibrating drum (m).

The amount of scatter which has been found to arise in levels of vibration from operation of vibrating rollers and other types of plant, has stimulated the use of a probabilistic approach to prediction. It is for this reason that the three different values of k are suggested. The predictor assumes a travel speed of approximately 2km/h, which is the middle of the range required by the Specification for Highway Works. If significantly different operating speeds are anticipated, then this should be accounted for by the square root relation described in Section 4.3.1.1.

During the start up and run down of vibratory rollers, the attenuation rate was found to be lower than that during steady state vibration. A regression analysis was therefore undertaken to develop a predictor for these transient stages of operation. The relation between the nominal drum amplitude and the ppv was the same as that during steady state operation. The following relation was determined:

$$v_{res} = k_t \sqrt{n} \left[\frac{A^{1.5}}{(x+w)^{1.3}} \right] \qquad (12)$$

where: k_t = 65, with a 50 per cent probability of the vibration level being exceeded;

k_t = 106, with a 33 per cent probability of the vibration level being exceeded;

k_t = 177, with a 5 per cent probability of the vibration level being exceeded;

Provision of separate predictors for steady state operation and the transient periods may enable operators to plan compaction processes appropriately in sensitive areas.

5 Groundborne vibration from piling works

5.1 Factors affecting vibration caused by piling

There exists a wide diversity of pile types and materials and many methods of installing the piles in the ground (see for example, Tomlinson, 1994). Methods of installing piles which are vibration free, or which generate relatively low levels of ground vibration, are available but these are not suitable for all situations. The use of techniques which generate significant vibration may be required in sensitive areas. The following review considers these piling methods and discusses the factors which may affect the level of groundborne vibration. A number of empirical prediction methods which have been proposed by other workers are presented and discussed in the light of new data acquired during the current research.

The intrusive nature of piling operations and the potential for damage to adjacent structures have meant that a large number of individual case studies has been undertaken and reported over many years. For example, Ferahian (1968) compiled a bibliography which illustrates that more than 60 years ago concern about pile driving close to a cathedral prompted a study of vibration (van der Haeghen, 1938). Consequently, many of the data have been collected and reported in a piecemeal fashion, often without the use of a structured or consistent approach, which makes them difficult to collate and analyse.

Reference to some of the published case histories illustrates the variability in the approaches adopted. Alpan and Meidav (1963), Clough and Chameau (1980) and Hiller and Crabb (1990), for example, gave details of the piles, driving method and ground conditions. However, many case histories are rather less well documented. The tabulated data appended to BS 5228 : Part 4 (BSI, 1992b) demonstrate the variability of the available literature. Similarly, Head and Jardine (1992) attempted to compile a database on piling vibrations from a range of organisations with the objective of assessing potential for annoyance or damage caused by piling operations. Of the 150 case histories supplied 'many records ... lack important information. A sixth of them were entirely without measurements of vibrations, so that opinions given appear to be based on subjective opinions or purely qualitative records'. The following review focuses on the more systematic work which has been undertaken to develop prediction methods.

The factors which affect the level of vibration at a point

in the ground, arising from any activity, are: the amount of energy transmitted into the ground by the source; the rate of attenuation of the energy as it propagates through the ground; and the distance of the observation point from the location at which the energy enters the ground. Piling methods differ from many other vibration sources in that the position of the source which transfers energy into the ground continually changes as piling progresses, since the tip gets progressively deeper and encounters different soil and the length of the pile shaft in contact with the ground increases as driving progresses. A further difference is that the actual energy source does not come into direct contact with the ground, except in the case of end-driven piles. Therefore factors which may need to be considered are the details of the pile and piling hammer or driver, the nature of the ground into which the pile is being driven and the distance from the pile to the measurement location. These factors are discussed below.

5.1.1 Distance from the source

The influence of attenuation on the prediction of vibration was discussed in Section 1.3. Attenuation is dependent upon the elastic properties of the ground and the distance travelled by the vibration. The latter is generally considered to be the more important over distances appropriate to civil engineering sources and therefore must be properly defined. For many vibration sources, the location at which the energy enters the ground is clear. In the case of piles, however, the situation is less easily defined. At the start of impact driving, all the energy dissipated into the ground enters the ground from the toe of the pile. As driving progresses, the length of pile below the ground surface increases so there is potential for energy to enter the ground from an increasingly long line source, as well as from the pile toe. The difficulty in defining the point where energy enters the ground, described in Section 1.2.2, has resulted in the development of two approaches to measurement and prediction. One approach is to use the distance measured along the ground surface from the point at which the pile penetrates the ground while the alternative is to specify the slope distance from the toe of the pile. This complicates comparison of some of the published data sets, in particular where the way in which the distance was measured is not specified.

Head and Jardine (1992) stated that the proposed preliminary guidelines on vibration prediction, based on Attewell and Farmer (1973), do not apply for horizontally measured distances closer than approximately 5m because of the complex wave propagation patterns in this zone. Additionally they observed that the horizontal distance may be an inappropriate parameter as the pile is driven to depth, and the distance from the pile toe to the measurement point may be more suitable. Many of the published predictors have been presented in terms of the horizontal distance from the point of entry of the pile into the ground for ease of application, although Wiss (1967) and BS 5228 : Part 4 : 1992 stress the importance of using the slope distance to the pile toe.

5.1.2 Method of driving

The method of driving governs both the nature and magnitude of the ground vibration. The British Standard relating to piling induced ground vibration (British Standards Institution, 1992b) describes three of the types of vibration introduced in Section 1.2.3: *continuous, transient and intermittent vibration*.

Vibratory pile drivers generate continuous vibrations. The vibrodriver clamps to the top of the pile and operate by means of one or more pairs of horizontally opposed contra-rotating eccentric weights. Operating frequencies are typically between 25 and 50Hz. The vibration reduces the shear strength of the soil close to the pile, thereby reducing the friction at the pile-soil interface. The combined weight of the pile and the driver cause the pile to be driven into the ground. Soils for which this and other techniques are suitable are given in the Piling Handbook (British Steel, 1997a).

The use of percussive piling hammers, such as drop hammers, generates intermittent vibrations. At large distances, the coda of the vibration from one impact may overlap with the arrival of vibration from the next impact, causing pseudo-steady state (Section 1.2.3) vibrations. Transient vibrations would occur from an isolated hammer blow, such as might occur during the initial setting of a pile. Vibration from dynamic compaction (Section 6.1.2) might be considered to fall between these two definitions, since an interval of several second exists between individual impacts.

During percussive piling, the hammer impact initiates a stress pulse in the pile which travels along the pile until it reaches the pile toe, at which point part of the energy is reflected and part transmitted into the ground. The relative proportions of the energy transmitted and reflected are governed by the relative acoustic impedances (specifically, the product of the density and the compressional wave propagation velocity of the material) of the pile and the ground and are given by Zoeppritz's equations (Zoeppritz, 1919). At the vertical pile/soil interface along the length of the pile, the angle of incidence will be close to 90° and so almost all energy is reflected and very little is transmitted into the ground. Consequently, even when the pile has been driven to considerable depth, the major part of the energy can be considered to enter the ground from the pile tip of a percussively driven pile (Attewell and Farmer, 1973). However, energy may also be transferred to the ground along the pile shaft through friction as the pile moves through the soil. This would generate a vertically polarised shear wave. It has also been suggested that flexure of the pile shaft may occur during driving, which can initiate vibration (Selby, 1991).

The different driving methods interact with the ground in different ways. The energy transmitted into the ground by percussive piling techniques gives rise to transient or intermittent vibration which is propagated at frequencies governed primarily by the characteristics of the ground. Soils do not possess resonant frequencies but generally behave as bandpass filters, possessing a limited range of frequencies within which most vibration energy propagates. These frequencies have been referred to as

characteristic frequencies (see Table 4). Conversely, vibratory pile drivers force the ground to oscillate at the operating frequency of the driver. Particularly high amplitude vibrations could arise if the driving frequency were to coincide with the preferred frequency of the soil.

A variation on vibro-driving is the use of resonant pile drivers which vibrate piles or casings at frequencies of up to 135Hz. The large amplitude vertical vibrations set up overcome the frictional resistance of the soil surrounding the pile allowing penetration by self-weight. Wiss (1967) reported tests with a 'sonic' pile driver operated at frequencies between 90 and 120Hz and a vibratory hammer operated between 16 and 21Hz, and compared the results with the vibration arising from impact driving. The vibration levels produced by the sonic driver were typically an order of magnitude smaller than those from a comparable impact driver, but continuously varied and occasionally attained levels of half those of the impact driver. The vibratory hammer generated levels similar to those from the impact hammer.

The magnitude of ground vibration is dependent upon the energy transmitted into the ground, which has been considered by many authors to be primarily governed by the nominal energy of the pile driver. This parameter has consequently been used for the development of predictive techniques for ground vibration problems relating to pile driving, with data commonly being presented as a function of the distance from the pile scaled by the theoretical energy provided by the driver. For the prediction of vibration from vibrodriving, the nominal energy term has been replaced by the nominal energy per cycle. These predictive methods are discussed in Section 5.2.

5.1.3 Pile type and dimensions

There is a wide diversity of pile types both in terms of the material from which they are made and their shape and dimensions. Some generic examples are: pre-cast concrete; cast *in situ* concrete (which may require a steel tubular casing to be installed and extracted, often by vibro driving); steel sheet; steel H section; and timber. The following discussion considers the effects on vibration generation of the length and cross sectional area of piles and the material from which they are made.

The length of the pile was considered to have an influence on the resulting vibration level by Prakash and Jain (1970). However, Lo (1977) reported vibration arising from driving pre-stressed concrete piles in two sections. The splicing of the upper section to the top of the pile did not affect the particle velocities measured on the ground surface.

Wiss (1967) considered sheet piles, wooden piles and H piles and reported that there was no significant difference in the vibration produced, all other variables being constant. Similarly, Whyley and Sarsby (1992) considered that, for a given energy, the variables relating to generation of ground vibration are hammer type, pile type and ground conditions, but their suggested predictors take no account of the details of the piles. Brenner and Viranuvut (1977) also concluded, from measurements made of piling in Bangkok Clay, that the size of the pile was not a significant factor in determining levels of ground vibration.

In contrast, Heckman and Hagerty (1978) considered that both the dimensions of the pile and the pile material are important in determining the energy transmission into the ground. A parameter which is considered to describe the ability of a pile to transmit a longitudinal impulse is the pile impedance, I, defined as (Poulos and Davis, 1980)

$$I = \rho v A_c \qquad (13)$$

where ρ is the density of the pile material, v is the compressional wave velocity of the pile material and A_c is the cross-sectional area of the pile. Heckman and Hagerty presented field data which indicated that as the impedance increased, the ground vibration decreased. A figure was presented to illustrate the variation of vibration level with pile impedance, the value decreasing from approximately 1.6 to 0.1J$^{-1/2}$/m for a range of pile impedances of 400 to 2200kg.s/cm. Head and Jardine (1992) commented on Heckman and Hagerty's work that, because of the wide range in ground conditions and the difficulties in accurately defining the energy levels of the drivers and the impedance of the piles, it is impossible to draw any general conclusions. However, Massarsch (1992) considered the findings of Heckman and Hagerty to be important since they indicate that, for example, a reduction in the pile impedance of 30 per cent could increase the ground vibration amplitude by a factor of 10. Also Dowding (1996) considered the research to be significant and suggested that differences in the pile impedance and the effects of the properties of packing between the pile head and the hammer were more significant than the penetration resistance of the ground in determining the ground surface vibration level.

Data given by Peck *et al* (1974) indicate that piles with similar dimensions but made of different materials can have impedance values which differ by a factor of ten. This, according to Heckman and Hagerty's work, would have a significant effect on vibration levels. Prakash and Jain (1970) considered that, under otherwise similar conditions, the minimum energy transmitted was by steel piles when compared with timber and concrete piles.

From Equation (13) it is apparent that, for piles of similar materials, the impedance is only dependent on the cross sectional area. A thin sheet pile of small cross sectional area would therefore have a lower impedance than a steel H pile and so, according to Heckman and Hagerty's work, might be expected to cause higher vibration levels than an H pile. This is supported by the data presented by Attewell and Farmer (1973) which show the sheet pile data to plot above the rest of the data. Lo (1977) and Attewell (1995) however, stated that smaller displacement piles should generally generate less vibration since less energy is required to drive them. Lo considered that the amplitude of vibration is proportional to the cross sectional area of the pile: a relatively *sharp* pile would penetrate further into the ground than a *blunt* one for a given impact energy. Consequently more energy would be expended in advancing the sharp pile, leaving less energy to propagate as groundborne vibration.

The preceeding discussion demonstrates that there are two effects to consider, both dependent upon the cross

sectional area of the pile but which work in opposition. A pile of small cross sectional area has a low impedance and therefore, according to Heckman and Hagerty, would result in a higher level of vibration than would a larger pile, for a given driving energy. However, the small pile would penetrate the ground more easily than a large pile and therefore a lower level of groundborne vibration would arise from the smaller pile than from the larger pile. This distinction has not been made in the literature and no data on the relative magnitudes of the two effects are available. At refusal, the penetration effect would not be significant and differences would be due solely to the impedance of the pile and to the relative acoustic impedances of the pile material and the ground (Section 5.1.4).

5.1.4 Ground conditions

The ground conditions can affect groundborne vibration arising from piling in two ways, by the interaction between the pile and the ground affecting the source level of vibration and by the effect of the ground conditions on the propagation of the energy away from the pile. The following discussion addresses the former effect; the second effect was discussed in Section 1.3. The energy transferred from the hammer to the pile remains approximately constant throughout driving (Rempe and Davisson, 1977), except where lower energies are used for the initial toeing in of the piles. There may also be some change in the energy through changes in the properties of the packing caused by the driving. In the simplest case, for a given pile and driver, changes in the vibration level must be dependent upon the relative amounts of energy used in advancing the pile through the ground and in causing recoverable (elastic) deformation of the surrounding soil. It is these elastic deformations which give rise to groundborne vibrations.

Luna (1967) presented data from measurements made on sand, gravel and clay to which curves were fitted. Although there was considerable scatter in the data, it was concluded that the vibration level generated was dependent on the soil conditions, type of pile, type and size of hammer, depth of pile penetration and distance from the pile. Luna also observed that vibration levels were affected by the penetration resistance and increased when dense strata or boulders were encountered. Furthermore, attenuation in clays appeared to be less rapid than in sands and transmission of vibration increased for higher ground water table

The most important factor governing the distribution of the available energy between these two components was considered by d'Appolonia (1971) to be the resistance of the soil to pile penetration. In stiff or dense soils a relatively large amount of energy is dissipated as elastic deformation of the soil and penetration is small. In easily penetrated soils, most of the energy is expended in advancing the pile, resulting in relatively low levels of groundborne vibration. This phenomenon has been observed by many other workers, such as Baba and Toriuim (1957) who recorded an increase in vibration when piles in sands and gravels reached a firm layer. Wiss (1967) observed similar behaviour and also pointed out

that the frequency content, as well as the amplitude, of vibrations propagating away from impact sources is determined by the type of soil.

Ciesielski et al (1980) observed a relation connecting vibration amplitude, the resistance to driving presented by the soil and the driver energy. The driver energy was found only to be important when the driving resistance was high. When resistance to driving was low, the vibration amplitude showed no increase with increasing driver energy. These authors also concluded that vibration only increased with decreasing pile penetration rate when the decreased penetration was due to increased end resistance and not when restraint was caused by changes in lateral friction. Similarly, Hosking et al (1988) carried out trials for driving steel H piles using a 1.5 tonne hammer with various drop heights and found that the major factor affecting vibration levels was the density of the sand into which the pile was being driven.

Attempts have been made to quantify the effect of the geology at the point of penetration on ground surface vibration amplitudes. Lo (1977) presented a series of curves for different hammer types showing that the ppv increases with blow count (ie. the number of hammer blows required to advance the pile over a specified distance). Whyley and Sarsby (1992) also reported that levels of vibration during percussive driving increased with increasing stiffness and/or density of the soil. Similarly, Mallard and Bastow (1979) observed that, under constant driving energy, increases in ppv occurred with increasing resistance, particularly in the horizontal components of vibration. It was suggested that this may either be as a result of the arrival of a shear wave from the toe combining with the surface wave; or when the driving gets harder, the pile whips in a horizontal mode so exciting vibrations in that plane.

Massarsch (1992) also observed a similar phenomenon. In a dense sand deposit, friction induced conical waves were considered to arise from energy transferred along the pile shaft. Where a stiff layer occurred at the pile tip, waves were generated from the pile tip and from the shaft due to flexing of the pile. Whilst the consensus of opinion appears to be that harder driving leads to higher levels of surface ground vibration, Greenwood and Farmer (1971) reported that data from three different types of pile driving in sands had shown that the effect of end resistance during driving was not significant when compared with the energy input. Similarly, the work undertaken by Attewell and his co-workers, while acknowledging the effect of penetration resistance, takes no account of the ground conditions in the most recently published predictions based on their work (Attewell, 1995). However, Attewell (1995) referred to Uromeihy (1990) who presented a range of scaling factors for prediction of vibration from a variety of pile types in several different types of ground.

A number of workers have attempted to correlate the ppv arising from piling with data acquired during site investigation. For example, Brenner and Chittikuladilok (1975) and Brenner and Viranuvut (1977) showed a relation between Dutch cone resistance and peak particle velocity, although the coefficient of correlation given in

the latter paper is not high. These data tend to contradict an earlier comment in the paper by Brenner and Viranuvut which states that the type of layer penetrated by the pile has little influence on the magnitude of vibration on the ground surface. Van Staalduinen and Waarts (1992) also considered the effect of the cone penetrometer value on the magnitude of particle velocity. Jongmans (1996) has proposed undertaking additional testing ahead of the main works as a basis for site specific vibration prediction.

5.2 Review of earlier work on the prediction of vibration from piling

The research undertaken by Luna (1967) presented data in terms of the energy ratio (ER). This was defined by Crandell (1949) as $ER = a^2/f^2$, where a is the particle acceleration and f is the vibration frequency. Crandell had observed that the risk of damage to a structure was dependent upon both a and f, which is equivalent to particle velocity, the parameter most commonly used in current vibration damage assessment (Section 1.2.1). Field data from a number of sites on different soils were presented graphically by Luna as energy ratio against distance but no general prediction equation was proposed.

Wiss (1967) presented graphically the maximum vibration levels expected from pile driving in wet sand, dry sand and clay. Wiss was among the first authors to present piling vibration data with an abscissa of the square root of energy divided by the distance, which was described as the scaled energy. This method has subsequently been widely adopted for presentation of vibration data. The distance used by Wiss was the *seismic distance* (defined in Section 1.2.2) from the pile toe to the point of interest.

In 1973, Attewell and Farmer proposed an empirically determined prediction equation which has become the basis for many subsequent attempts at prediction. Vibration data were collected from seven sites with different pile and hammer types and different soils. The measurements were made over a range of distances from 1m to 40m for the full penetration sequence and various nominal energy inputs. Results were presented for vertical components of vibration which yielded a prediction of ground surface peak particle velocities for all driving methods and pile types given by:

$$v_v = k \left[\frac{\sqrt{W}}{r} \right]^{\beta} \qquad (14)$$

where v_v is the peak particle velocity (*vertical* component) (in mm/s);

W is the source energy per blow (or per cycle) (in J);

r is the radial distance between source and receiver (in m);

k and ß are empirically determined constants

Attewell and Farmer (1973) assigned a value of unity to ß and what has more recently (Head and Jardine, 1992; Whyley and Sarsby, 1992; Attewell, 1995) been considered a conservative value of 1.5 to the parameter k. This expression implies that the particle velocity is proportional to the square root of the energy in the system. The power of -1 in the distance term could be interpreted as indicating that attenuation is dominated by body wave effects, for which theoretically the particle velocity attenuates according to r^{-1}. It could also be inferred that, if surface waves dominate, for which the ppv attenuates according to $r^{-\frac{1}{2}}$, then a significant proportion of the attenuation must be due to other effects, such as internal damping. Particle trajectories determined by Attewell and Farmer indicated that the motion was similar to the retrograde elliptical motion characteristic of Rayleigh waves. It is therefore likely that the latter explanation is the better.

The distance r is described by Attewell and Farmer as the radial distance, although it is not clear from the paper whether this is measured horizontally from the pile axis or obliquely from the pile tip. Later work presented by Attewell (for example Attewell *et al*, 1992a, b) uses the horizontally measured distance so it is assumed here that this is also the case for the 1973 paper. Expressions of the form of Equation 14 are now widely used in the presentation of data and the development of predictors, with much effort having been concentrated on the determination of the value of k. Additionally, some authors have considered the indices of the distance and nominal energy terms and these are discussed below.

Using data from a large number of field measurements from many sites, Uromeihy (1990) determined a number of different values of k and ß (Equation (14)) for different combinations of hammer, pile and ground conditions. In addition, Uromeihy used the resultant ppv in place of the vertical component ppv. Based on further analysis of the data from Uromeihy (1990) and other sources, Attewell *et al* (1991) found that, rather than the vibration level decaying steadily with increased distance from the pile at all distances, maxima occurred at typically 10m (measured horizontally) from the pile. Attewell *et al* (1991) suggested that this effect may be a result of superposition of surface waves, caused by lateral movements of the pile at ground surface level, and body waves emanating from the pile toe. One implication of this is that any prediction method which has been developed from data which include distances of less than 10m could be unreliable if the predictive relations were extrapolated beyond the distance from which the data were acquired. The work described by Uromeihy (1990) presented models based on data acquired at horizontal distances from the pile of typically between 1m and 20m. It would seem that simple analyses of these data might result in significant errors in predictions, particularly if relations were extrapolated beyond the limits of the data. It was therefore suggested by Attewell *et al* (1991) that Attewell and Farmer's (1973) relation is only valid for distances greater than 10m. Attewell (1995) suggested that because of the conservatism inherent in Equation (14), a more realistic equation from the same data would have a value of k of 0.75 to predict the zero-to-peak *resultant* ppv.

Head and Jardine (1992) also considered that a value of k of 1.5 in Equation (14) is conservative and suggested that, for values of $\sqrt{W}/r$ in the range 1.0 to 30J½/m, the

expression could be used for preliminary assessment with the particle velocity being specified as the *simulated* resultant particle velocity (Section 1.2.1). Furthermore, Head and Jardine proposed that *k* has a value of between 0.1 and 1.5 (for *W* in joules and *v* in mm/s), depending on the ground profile; and ß varies between 0.8 and 1.5, depending on the soil characteristics. However, no guidance was provided on the how to select appropriate values of *k* and ß.

Attewell *et al* (1992a and b) concluded that, because of observed curvature in the data field, a linear log-log relation was not strictly valid and quadratic log-log curves were introduced. Families of curves were calculated based on the best fitting curves to the data sets for impact hammers and vibrodrivers. These enabled the prediction of particle velocities for a variety of drivers at distances of up to 20m. While this is an adequate range of distances to assess the majority of cases where damage is a possibility, vibration levels at 20m are still well within the range which may be intrusive. The predictions could not therefore be reliably used to predict the more prevalent problem of perceptible disturbance due to vibration effects. Additionally, these papers by Attewell *et al* only consider the variables ppv, standoff distance and nominal driver energy for, separately, vibrodrivers and impact hammers. No account is taken of the ground conditions or the pile type. Furthermore, the tables and graphs presented by Attewell *et al* indicate a progressive attenuation over distances between 2 and 20m; there is no indication of a rise preceding the decay with distance reported by Attewell *et al* (1991).

For practical convenience, predictive curves and equations proposed by Attewell *et al* (1992a and b) were specified in terms of the horizontal distance from the pile to the measurement location. In cases where the majority of the energy entering the ground during pile driving does so at the tip of the pile, this approach must present some degree of error, particularly since most of the data were determined at horizontal distances of no more than 20m. The source becomes more distant from each measurement location as driving progresses and in some cases the depth may exceed the horizontal distance. Furthermore, it is possible that the amount of energy transmitted into the ground by vibration will increase as driving gets harder (see for example, Mallard and Bastow, 1979). Consequently, the peak vibration level recorded at each geophone may occur when the distance to the actual source is significantly greater than the minimum distance to the pile.

The use of a best fit line based on field data is unsatisfactory as a predictor since, by its definition, half of the data will be at vibration levels greater than those predicted. Also, without additional information, this approach would not indicate by how much vibration levels might exceed those predicted. Furthermore, if the data acquisition system used to record the data on which the predictor is based is only capable of sampling short discrete sections of data, the level of this best fit line will be dependent upon the amount of data sampled during periods of relatively low amplitude vibration and on

whether the actual peak amplitudes were captured. This could be a problem particularly for vibrodriven piles, where the vibration is continuous and also may contain starting and stopping transients. Attewell *et al* (1992a and b) therefore recommended that a curve one-half a standard deviation above the mean is used as a predictor. This would predict a vibration level that there was a 31 per cent chance of being exceeded. A more conservative approach would be to use one standard deviation above the mean, which would reduce the chance of exceeding the predicted level to 16 per cent. Combining this with the quadratic interpretation discussed above, expressions of the following form were presented, with different values of k_1, k_2 and k_3 given for impact hammers and vibro drivers as shown in Table 5.

Table 5 Values of constants for use in the predictive equation presented by Attewell et al (1992a)

	Vibrodrivers		Impact hammers	
	Half standard deviation above mean	One standard deviation above mean	Half standard deviation above mean	One standard deviation above mean
k_1	0.213	0.038	0.296	0.073
k_2	1.64	1.64	1.38	1.38
k_3	0.334	0.334	0.234	0.234

$$\log v = -k_1 + k_2\log\left[\frac{\sqrt{W}}{x}\right] - k_3\log^2\left[\frac{\sqrt{W}}{x}\right] \tag{15}$$

where *W* is the hammer, or driver, energy (in joules, or joules per cycle);

x is the horizontal distance (in metres).

More recently, Attewell (1995) has presented these expressions, together with simpler predictors in the form of Equation 14. The latter are presented in Table 6, together with similar expressions adopted by the BSI (1992b) for the prediction of vibration from piling.

Whyley and Sarsby (1992) adapted the power law approach to prediction of vibration arising from impact piling, suggesting that the value of *k* Equation 14 is dependent upon the ground conditions. A prediction method was proposed which was based on data from sheet piling but which can also be used for other types of pile driving. A graphical presentation enabled predictions of ppvs to be made for piling in three different types of soils. The parameter *k* was assigned values ranging from 0.25 to 1.5, depending upon the soil type, based on horizontal distance from the pile. A similar method has been adopted by Eurocode EC3, Chapter 5 (CEN, 1998) and is included in the latest guidance from British Steel (1997b). This assigns values of between 0.5 and 1.0 to *k* for impact piling, dependent upon the soil type. For vibrodriving, a value of 0.7 is suggested for all soil conditions.

All the prediction methods described above essentially attempt to enable preliminary predictions of vibration to be made by desk study alone. An alternative approach was proposed by van Staalduinen and Waarts (1992), who used

Table 6 Simple predictors for preliminary estimation of vibration from piling proposed by *Attewell and †BSI

	Vibrodrivers	*Impact hammers*
For a high level of confidence that predicted values will not be exceeded[*]	$v_{res} = 1.8 \left[\dfrac{\sqrt{W_c}}{x} \right]$	$v_{res} = 1.5 \left[\dfrac{\sqrt{W}}{x} \right]$
For more probable levels of vibration[*]	$v_{res} = 1.0 \left[\dfrac{\sqrt{W_c}}{x} \right]^{0.95}$	$v_{res} = 0.76 \left[\dfrac{\sqrt{W}}{x} \right]^{0.87}$
Vibration levels unlikely to be exceeded significantly in most cases[†]	$v_v = 1.0 \left[\dfrac{\sqrt{W_c}}{r} \right]$	$v_v = 0.75 \left[\dfrac{\sqrt{W}}{r} \right]$

v_{res} is the resultant ppv (mm/s); v_v is the vertical component ppv (mm/s); W is the nominal energy of the hammer (J); W_c is the nominal energy per cycle (J); x is the distance measured along the ground surface; r is the slope distance from the pile toe.
[*]Attewell (1995)
[†]BSI (1992b)

data from cone penetration tests to characterise the ground conditions, from which vibration predictions were made. The authors accepted that this technique would be less reliable than using site specific vibration measurements but observed that it would also be less expensive. Given the variability of the data reported by other authors and the discussion over attenuation rates and values of k, this approach would appear to offer a potentially more reliable prediction method. The prediction was based on the attenuation equation proposed by Mintrop (1911) (Equation 5).

The vibration level, specified at a reference distance of 5m, was determined by van Staalduinen and Waarts to be dependent upon the total driving resistance (F_d). A relation was therefore determined between F_d and the cone penetration, which exhibited a better correlation if the vibration level was specified in terms of particle accelerations rather than velocities. The proposed model, while appearing to be successful, had only been developed on one site and required further validation.

Another approach has been presented by Jongmans (1996) who observed that empirical prediction methods can yield erroneous vibration assessments, in particular because the relations are based on the assumption of a half-space which ignores the effect of local geological conditions on the amplitude of the ground motion. Parametric studies were cited which demonstrated that the amplitude, frequency content and duration of the signals are greatly dependent on the dynamic properties and the thickness of the upper soil layer.

Jongmans observed non-monotonic attenuation and suggested that this may result from the interference between surface and body waves. In the near field, the surface waves are not well developed, whereas at greater distances the different propagation velocities of the surface waves and P-waves mean that the different wave types no longer coincide. There occurs a point at which the developed surface wave and the P-waves combine to give a maximum.

Jongmans' method requires the separate determination of a source function and a propagation function in order to reconstruct the ground motion at any distance. The technique requires a seismic investigation of each particular site to determine the geometry and dynamic properties of the ground, in particular the nature of any layers, from which the propagation function is determined. The vibration source, assumed to be at the pile toe, is dependent upon the pile type, driving method and ground conditions at the toe. Jongmans illustrated an example where the proposed technique was successfully tested and he advocated the assembly of a database so that the technique can be applied to other sites. Unless this database technique was used, it would require a relatively expensive initial investigation on each site to predict vibration levels which would be generated by the main works.

To summarise, a number of empirical prediction methods have been proposed. Those in the form of the Attewell and Farmer (1973) equation (Equation 14) consider the nominal energy (W) of percussive pile drivers and attempt to fit the best curves to the data. This has resulted in a series of different scaling factors to apply to basically the same equation. For prediction of vibration from vibrodriving, W has ben replaced by the nominal energy per cycle (W_c). Some authors have made provision for piling in different soils by the inclusion of different scaling factors depending upon the ground conditions. The approaches taken by van Staalduinen and Waarts (1992) and Jongmans (1996) attempt to quantify the effects of other parameters to adapt predictions to specific site conditions. However, both of these techniques require further validation. A further approach which could be taken to this work is through the development of theoretical models using finite element modelling. Such studies are currently being undertaken elsewhere (eg Ramshaw *et al*, 1998).

5.3 Acquisition of field data from piling

Piling data for the current study have been acquired from 9 sites, covering 12 different piling operations, the details of which are summarised in Tables 7 and 8. Given the large number of variables associated with piling, it cannot be considered that there are sufficient new data for all the factors discussed in Section 5.1 to be fully addressed, nor

Table 7 Details of percussive piling sites

| Site | Geology | Pile details | | | Hammer details | |
		Type	Length	Section	Type	Nominal energy (J)
A13 East Rail Crossing	0-12m: soft silty clay; 12-18m: terrace gravel; 18-24m: dense fine gravel.	Precast concrete	24m	350mm square	BSP HH5LD hydraulic hammer	24520 (5t x 0.5m)
M66 Hollingwood Railway Bridge	0-0.5m: made ground; 0.5-24m: firm sandy clay.	Steel sheet	20m	Larssen 25W	BSP HA357	24520 (5t x 0.5m)
A47 Church Road Interchange	0-2m: firm silty clay; 2-14m: loose silt; 14-17m: very dense sand + chert gravel; 17-30m: very stiff silty clay	Steel H	13m (1st drive) 28m (total)	UBP 305x305x186	Banut 700	24520 (5t x 0.5m)
A548 Dee Crossing (Eastern approach)	0-2m: silt; 2-23m: medium dense fine sand.	Precast concrete	8m	360mm square	Not recorded	44136 (5t x 0.9m)
M25 Environmental barrier	Made ground	Casings	4.35m	610mm dia.	BSP 1D17	15000
Dudley Vale, Radstock	Made ground: colliery waste.	Steel sheet	8m	Not recorded	BSP HH 1.5DA	18149

Table 8 Details of vibratory piling sites

| Site | Geology | Pile details | | | Vibrodriver details | | | |
		Type	Length	Section	Type	Power (kW)	Frequency (Hz)	Energy per cycle (kJ)
A548 Dee Crossing (Western approach)	0-1m: very soft silt; 1-5m: firm to stiff glacial till; >5m: weak mudstone.	Casings	9m	Not recorded	PTC 25H2	186	28	6.64
A11 Foxes Bridge	0-3m: very dense sand; 3-4.5m: grade IV chalk; 4.5-6m: grade III chalk; 6-15m: grade II chalk.	Casings	14m	900mm diameter	PTC 25H1	144	29	4.97
Second Severn Crossing approach	0-1m: made ground; 1-4m: stiff clay; 4-14m: soft to very soft clay; 14-16m: firm to stiff marl.	Casings	15.5m	1050mm diameter	PTC 50H3	290	27.5	10.55
Derby Southern Bypass, Trent Bridge	0-2m: stiff sandy clay; 2-6m: medium dense gravel; 6-9m: mudstone and siltstone.	Steel sheet	8m	Not recorded	PTC 13HF1	163	38.3	4.26
Dudley Vale, Radstock	Made ground: colliery waste.	Steel sheet	8m	Not recorded	ICE 14RF	213	38.3	5.56
		Steel sheet	5m	L40	ICE 328SH	46/55	46.7	0.99/1.17

is it possible to derive a wholly new prediction method. However, the data which were acquired contain possibly more detail than was available to many of the earlier researchers because of the nature of the acquisition equipment used. For example it has been possible to acquire data for the entire duration of driving for some piles, for which triaxial measurements have been made at five locations simultaneously at distances ranging from the order of metres to over 100m. Other workers have been restricted by more limited sampling periods or smaller ranges of distance. The new data therefore provide a means of validating the existing predictors.

During acquisition of data from impact piling, the depth of the pile toe beneath the ground surface was recorded, by reference to the gradations on the pile used by the contractors to determine the blow count. This was not possible for vibrodriving, since the piles were not marked in this manner. The amount of data acquired from each site

was variable, depending largely upon the rate of progress of piling. In all cases it was attempted to acquire data from driving a representative number of piles.

The data collected by the Transport Research Laboratory have been supplemented by further analysis of the database established by Uromeihy (1990). These data were available in the form of peak component and true resultant particle velocities for horizontal distances of typically 1m to 20m, at specified depths of penetration. Details of the hammer, pile and ground conditions for most sites were also tabulated. The inclusion of these data has made a valuable contribution to the current research.

5.4 Validation and development of piling vibration models

The first stage of validating the prediction models and interpreting the data was to plot the resultant ppv recorded at each site against the distance from the pile. On to these plots were added some of the predictors which have been described above; these are presented in Figure 29. The plots demonstrated that the existing prediction methods were not reliable. In particular, the A47 data exceeded even the supposed conservative upper bound predictor of Attewell and Farmer (1973). Attempts were made to establish the causes of the variation and to develop and improve the predictors.

The efficacy of the predictors appears to reflect the distances over which the data from which the predictors were derived have been acquired. Few previous data have been presented which cover as large a range of distances as that for the new data. Attewell and Farmer (1973) derived their relation from data covering distances from 1m to 40m, but the prediction tended to over estimate vibration

close to the pile. The more recent data, from Attewell *et al* (1992a and b), focused on horizontal distances of typically 1m to 20m. Comparison of the 1973 relation with these data would indicate that the former was overly conservative. The predictors developed from the later studies are less conservative close to the pile.

Although most authors have adopted similar approaches for the prediction of vibration from impact and vibratory driven piling, the data showed these activities were sufficiently different that they should be analysed separately. These analyses and the interpretation of the data are described in the following sections.

5.4.1 Analysis and interpretation of data from impact piling

The data from the A47 site provided a valuable basis for the majority of the interpretation work for impact piling. The piles had a total length of 28m and were driven in two sections. This enabled the effect of how the distance term is specified to be addressed. Additionally, the pile encountered changing ground conditions over the course of the drive, which enabled the effect of the change in driving resistance to be investigated while all other parameters relating to the pile and hammer remained constant.

5.4.1.1 Specification of the distance term

Plotted as a single group of data on logarithmic axes of resultant peak particle velocity (ppv) against horizontal distance, the scatter of the A47 data covered a range of approximately an order of magnitude at all distances. The data set was therefore subdivided into groups dependent upon the depth to which the pile had been driven, arbitrarily grouped in increments of 5m. The data were
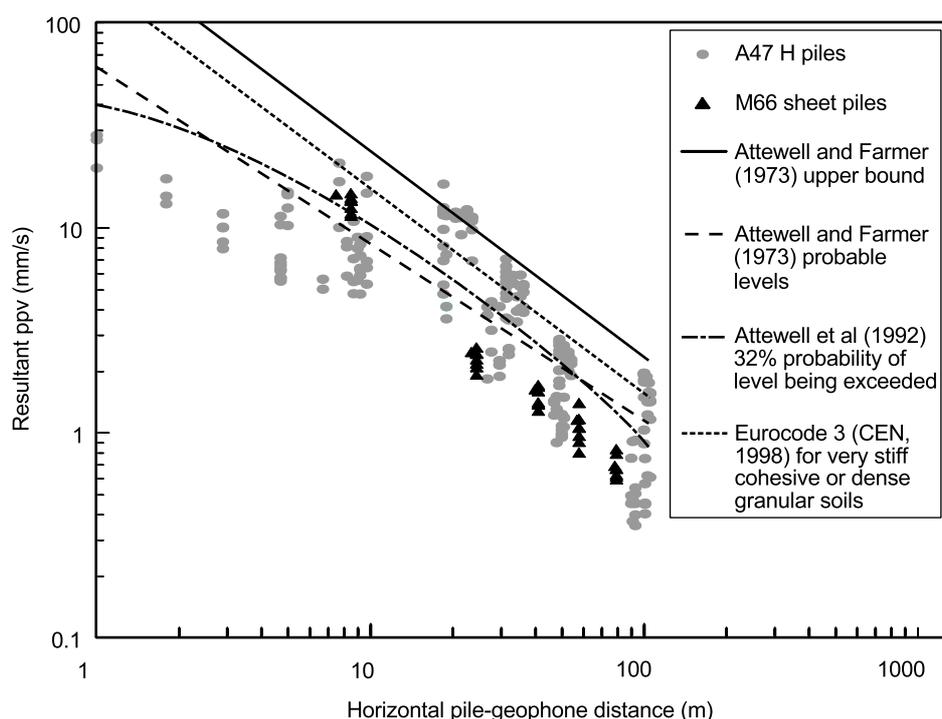
Figure 29 Current piling predictors plotted with TRL data from two sites

then replotted against horizontal distance from the pile on logarithmic axes (Figure 30a). This revealed that, at distances in excess of approximately 20m, the data separated into a series of sub-parallel groups, the vibration level being greater for greater penetration depths. The lowest levels at any distance occurred while the pile toe depths were smallest. In part, at least, this may have arisen because lower driving energy was used for the toeing-in operation, which drove the pile to a depth of 4m. However, for the subsequent 24m of driving, the driving energy remained constant for all depths.

At horizontal distances of less than 20m, the pattern broke down, such that the lowest levels of vibration no longer corresponded with the pile toe being closest to the surface.

Figure 30 Ground vibration data from the A47 piling site plotted against (a) horizontal distance from the pile and (b) slope distance from the pile toe

This suggested that the transfer of the majority of energy into the ground may take place at the toe of the pile. Replotting the data against the slope distances from the pile toe to the measurement positions sorted the data into a series of bands, indicating an increase in ppv as the pile was driven deeper (Figure 30b), for the whole range of distances.
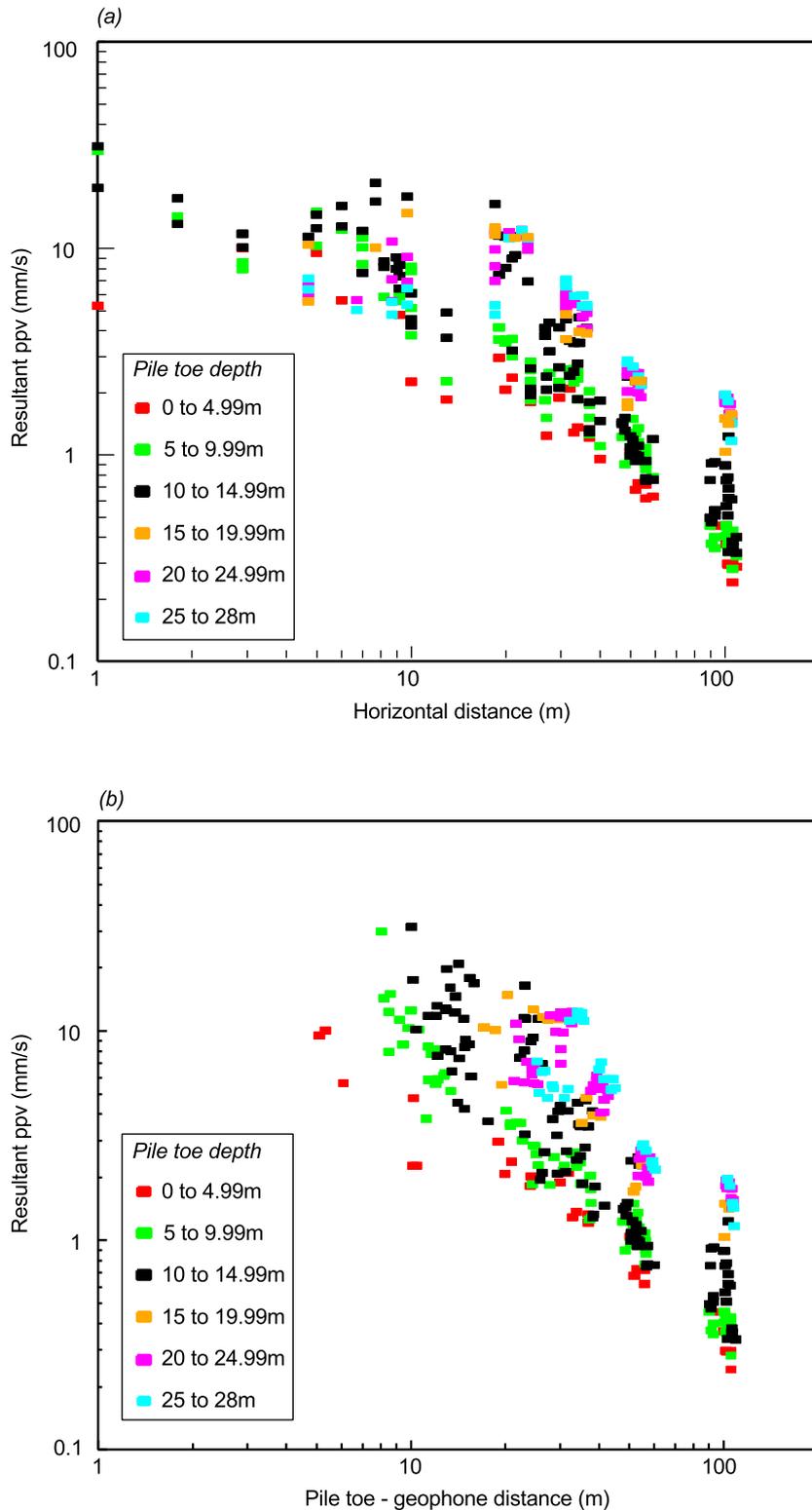
Looking at the data in more detail revealed two further aspects to the trends.

i When plotted against the slope distance to the pile toe on logarithmic axes, there existed a curvature in the data, which was concave upwards. The degree of curvature increased as the pile was driven deeper.

ii For each group of data, from the different toe depths, for pile toe depths in excess of 10m, the maximum ppv did not occur at the minimum distance from the pile. The data from closest to the pile were lower than the maximum ppvs observed for each group. That is, with increasing distance from the pile, the vibration level increased initially and then decreased. This effect was observed for data plotted against the slope distance to the pile toe. When plotted against the horizontal distance, where sufficient resolution was available in the data, there was an initial decrease in vibration level, followed by a rise and then continued decrease with increasing distance.

These two issues are discussed below.

i Concave-upwards curvature in the data field and the effect of pile toe depth

As a pile is driven into the ground, there are two effects to consider which may influence the magnitude of vibration observed at the ground surface. Firstly, as the pile toe is driven to progressively greater depth, energy transmitted

into the ground at the toe has to travel along an increasingly long path to reach the measurement location at the ground surface. Assuming a point source and a homogeneous half space, the energy would radiate as a sphere of increasing radius. The ppv would therefore attenuate in inverse proportion to the length of the direct travel path from the pile toe to the measurement position. Use of the horizontal distance from the pile for the specification of attenuation is therefore reasonable at larger distances, but close to the pile this introduces significant error.

Comparison of SPT data from the site investigation data and a pile driving record from the A47 site (Figure 31) indicated that the penetration resistance of the ground increased with depth for depths greater than approximately 12m. Combining this with the observation that, for a constant distance from the pile toe, the vibration level increased as the pile toe was driven deeper, suggests that the ppv is in some way related to the penetration resistance of the ground. The nominal energy input of the piling hammer was constant for depths greater than 4m, so changes in energy input to the pile could not have been responsible for the increasing vibration level. It follows that, if the energy input to the pile is constant, as the distance penetrated by the pile per hammer blow decreases, there must be more energy available to cause elastic deformation at the pile tip. There will also be some increase in energy transmission at the pile toe because the acoustic impedances of the pile material and ground will converge as the pile encounters increasingly stiff and dense geology.

To investigate the relation between the resultant peak particle velocity, at a constant slope distance from the pile toe, and the depth of the pile toe, the data from the most distant geophone were considered. These were at horizontal distances from the pile of between 100m and 109m. Using the data from the largest distances minimised
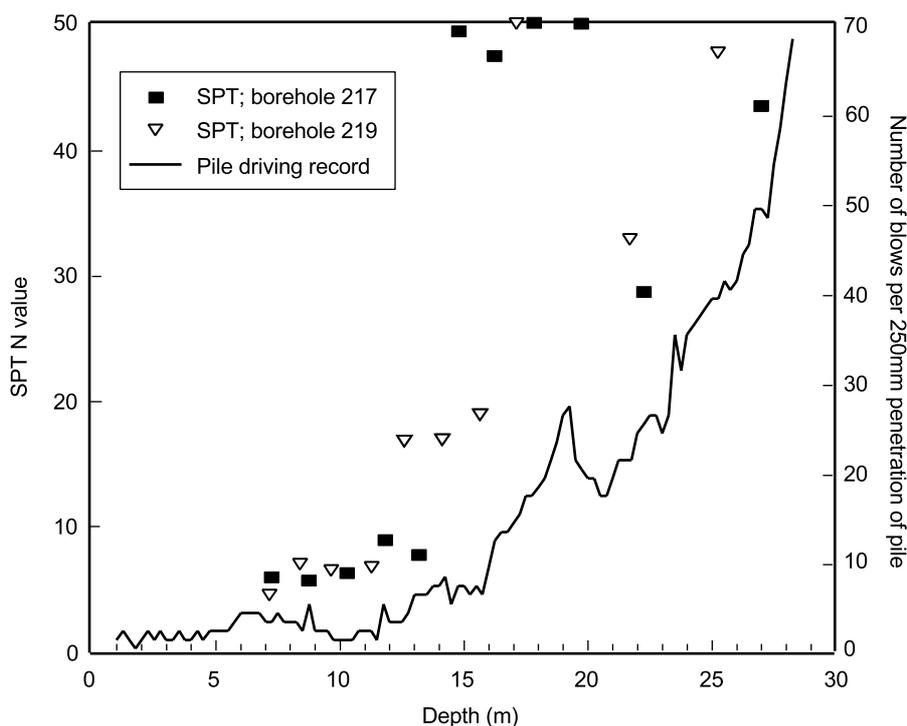

Figure 31 SPT and pile driving data from the A47 piling site

the effects of the changing depth of the pile toe, and the differences in horizontal distance, on the distance from the pile to the geophone. Plotting the resultant ppv against the depth of penetration of the pile revealed a clear relation between these parameters (Figure 32).

For the subsequent analyses, it would be necessary to apply the relation determined between the resultant ppv and the pile toe depth to the data from the geophones at all distances from the pile. This was achieved by analysing the data in terms of decibels (dB), referencing the particle velocity to the conventional reference ppv of 10^{-5} mm/s. Conversion from ppv in mm/s ($ppv_{mm/s}$) to dB re. 10^{-5} mm/s (ppv_{dB}) is achieved using the expression;

$$ppv_{dB} = 20 \log_{10} \frac{ppv_{mm/s}}{10^{-5}} \qquad (16)$$

Using a decibel scale enabled the relation derived from the data at the most distant geophones to be applied to all the data, since a change in the number of decibels always produces the same ratio change in the vibration level. For example, an increase of 3dB always equates to a doubling of energy, which represents an increase of $\sqrt{2}$ in ppv, irrespective of the absolute magnitude of the ppv.

A least squares best fit line was determined for the relation between the resultant ppv and the depth of the pile toe for pile toe depths greater than 4m. At lesser depths, the vibration level had remained approximately constant as the depth had increased. The following relation was determined:

$$v_{dB} = 0.74 \; L + 86.77 \qquad (17)$$

where v_{dB} is the resultant ppv (dB re. 10^{-5} mm/s);

L is the depth below the ground surface of the pile toe (m).

While there was some degree of scatter, the relation was useful for progressing the analysis. It should be pointed out that it is not intended that this expression is used in making vibration prediction on any other site. The analysis described here was undertaken with the objective of understanding the processes which determined the trends observed in the data. It is likely that this relation is unique for the particular combination of pile, driving method and ground conditions at the A47 site.

To continue the analysis, a set of initial conditions was selected from which to develop a model. Initial levels of vibration at a number of different horizontal distances (x) from the pile were assumed, based on the A47 data, for an initial pile toe depth (L_0) of 5m. A range of pile toe depths, L, were also selected, for which to calculate the ppv at the selected values of x. Assuming an attenuation rate for the particle velocity of r^{-1} (where r is the slope distance from the pile toe), the ppv was calculated for each of the chosen values of x and L, initially neglecting the effect of the increase in ppv caused by the stiffer soil encountered at increasing depth. These values of ppv were then converted to the decibel scale, and the effect of the ground described by Equation 17 was added.

The calculated ppv was plotted against the slope distance from the pile toe and against the horizontal distance from the pile. Although there are some similarities with the field data, the concave upwards trend observed in the field data when plotted against the pile toe depth was not replicated. Furthermore, the model did not provide the maxima observed in the field data at some distance from the pile during the later stages of driving. This latter aspect is discussed in the following paragraphs.
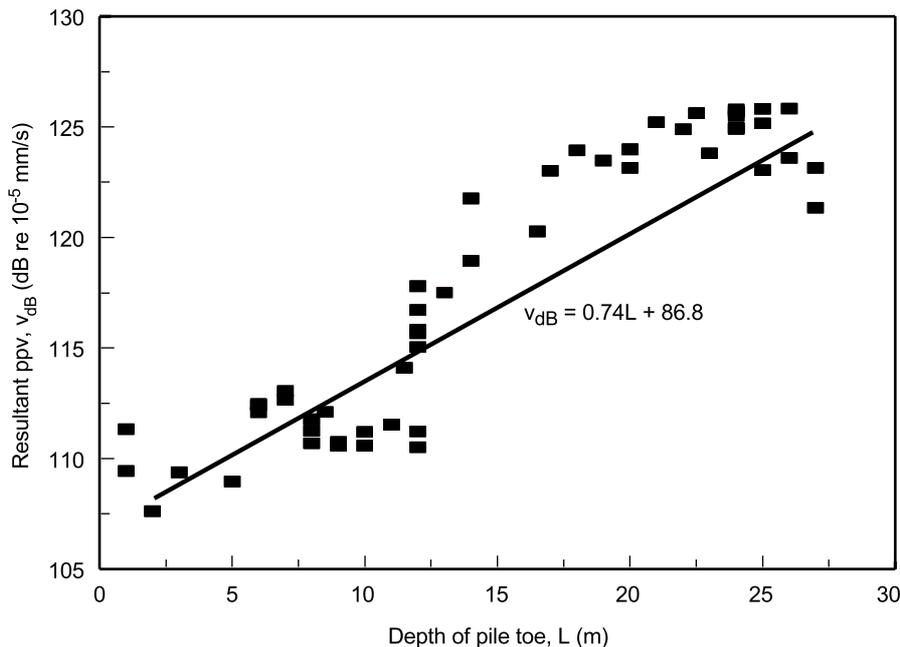

Figure 32 Relation between vibration level at a distance of 100m or more from the pile and the pile toe depth at the A47 piling site

ii Variation of vibration level with distance close to the pile

The following discussion describes the investigation undertaken to attempt to explain the detailed distribution of ppvs, in particular those close to the pile. For the purpose of this discussion, "close" is defined as the range of distances measured along the ground surface which is less than the embedded length of the pile. Figure 30 illustrates the data from the A47. It can be seen that the vibration level decreases away from the pile over the first few metres and then increases again before attenuating in the manner conventionally reported.

Where data were recorded from a drive for which the closest geophone was at a distance greater than perhaps 5m, which was the majority of the data, the initial decay in ppv over the distances from 1m to 3m was missed and the maximum level of vibration recorded was often not at the geophone closest to the pile. The A47 works had required that the close-in geophones were not mounted in the natural ground, but on compacted fill material which was serving as an operating platform. This may, therefore, have explained the low level of vibration. In addition to the main vibration event, there were also higher frequency, lower amplitude events from each hammer impact which were noticeable as distinct events. These events, which are explored in more detail later in this section, exhibited a monotonic decay over the whole range of distances, which suggests that the low levels close to the pile were real. Furthermore, vibration data from numerous sites reported by, for example, Uromeihy (1990) and Oliver and Selby (1991) also demonstrated that close to the pile there often occurs an initial rise in the ground surface vibration level, followed by a continuous decay with increasing horizontal distance.

Attewell *et al* (1991) and Jongmans (1996) attributed this non monotonic decay effect to interaction between vertically polarised shear waves arising from the pile shaft and compressional waves arriving at the ground surface from the pile toe. The shear waves travel more slowly than the compressional waves, so the interaction between the two takes place at a distance from the pile determined by the relative speed of propagation of the different wave types and the depth to the pile toe. Since compressional waves typically travel at approximately twice the speed of shear waves, the maxima in the data would be expected at a distance of approximately $L / \sqrt{3}$, where L is the depth of the pile toe beneath the ground surface, if it is assumed the shear waves are initiated at the ground surface.

The above explanation generally fits the distribution of vibration observed at the A47 site, in that the position of the maximum vibration occurs at a greater distance as the pile toe is driven deeper and this position approximately coincides with the $L / \sqrt{3}$ prediction. Furthermore, rather than simply an initial rise, followed by a decay, where data were available from very close to the pile, there was an initial attenuation interrupted by maxima at between approximately 8m and 20m, measured horizontally, depending on the depth to the pile toe. The shear and body wave model would predict this distribution, since some attenuation should be observed before the point of constructive interference.

To attempt to verify that the maxima in the distribution of ppvs arose through superposition of the two waveforms, the time histories were inspected. If the model were valid, for shorter distances than those where the maximum ppv occurred, the main vibration event would be preceded by another wave arrival *ie* it should be possible to identify separate arrivals for the wave arriving from the toe and that from the shaft. This was not observed and no evidence was found on the time histories to support the model.

There were, however, observed to be high frequency signals which, in general, arrived ahead of the main vibration event. At most distances, the magnitude of these was more than an order of magnitude less than the main vibration event. Close to the pile, however, the amplitude was more significant and at a horizontal distance of 1m, the high frequency event was superimposed on the main arrival. The following discussion describes the investigation undertaken to attempt to determine the origin of these high frequency events.

The origin of the high frequency signals was initially thought to be from reverberation of the stress pulse within the pile. For a steel pile with a compressional wave velocity of 5700m/s and a length of 28m, the frequency would be 102Hz. This was very similar to the observed high frequency. Similarly, Hiller and Crabb (1990) reported energy within the range 158 to 249Hz for 12m steel H-piles driven by impact piling, which was attributed to this effect. Further investigation of the field data, however, contradicted the hypothesis. The piles at the A47 were driven in two sections, the first having a length of 13m, to which was welded the upper section following driving of the lower length. Comparison of the spectra from similar horizontal distances for the two stages of driving revealed that the spectral peak was at a similar frequency for both pile lengths.

Similar high frequency events were recorded on the traces from the other impact piling sites studied. On two of these sites, the A13 in Essex and at the Dee Crossing, square section precast concrete piles were driven. The lengths of the piles on these sites were 24m and 8m, respectively. Although some variation in the compressional wave propagation velocity of the piles might be expected, the high frequencies associated with the shorter piles were 90Hz, while driving the longer piles gave rise to 190Hz events, which is contrary to what would be expected for reverberation of a stress pulse. It was not possible within this study to identify the source of the high frequency components of vibration.

The maxima observed in the distribution of vibration levels do not arise through interaction of the high frequency vibration and body waves from the toe, since the high frequency components travel much faster and therefore arrive before the main event except for within a few metres of the pile. On one file from the A47 the peak amplitude high frequency events coincided with the peak of the low frequency vibration. This only occurred at a horizontal distance of only 1m from the pile. At such a close range, the high frequency had a magnitude which was large enough (approximately 10mm/s) to make a significant contribution to the overall signal because the

magnitude of the main signal levels off at close range.

Further investigation of the source of the local maximum vibration level considered the work by Nakano (1925), cited by Ewing *et al* (1957), which showed that Rayleigh waves do not appear close to the epicentre of the source, but first appear at a horizontal distance (x) from the epicentre of the source, where:

$$x = \frac{c_r h}{\sqrt{c_p^2 - c_r^2}} \tag{18}$$

where h is the depth of the source;

c_r is the Rayleigh wave propagation velocity; and

c_p is the compressional wave propagation velocity.

Returning to the A47 data, for which a surface wave velocity of 220m/s and compressional wave velocity of 1650m/s, were determined from first arrivals on the time histories, Equation 18 predicts that surface waves would first appear at horizontal distances from the pile of 0.13 of the pile toe depth. Even for a toe depth of 28m, this would predict the presence of surface waves at distances as close as 3.64m, *ie* they would be present on all the time histories acquired during this study, with the possible exception of the end of driving where the closest geophone was 1m from the pile (File A47B15). Furthermore, the P-wave velocity at depth may be greater than that at the surface because of the stiffer materials encountered. This would have the effect of moving the predicted first appearance of R-waves still closer to the pile.

Inspection of the time histories in file A47B15 showed that a low frequency event does occur ahead of the main event on most of the files from the A47 (Figure 33). This appears approximately 0.4 seconds before the start of the main event on the geophone at 1m from the pile throughout driving (ie for toe depths of approximately 6m to 12m). This event is, however, also present on other files and maintains a common separation from the main event of approximately 0.4s for all distances.

There is no indication from the time histories to suggest that the local maximum in the plot of peak particle velocity against distance arises through interference of vibration from two sources. Although there are indications on the time histories of at least two events associated with each hammer impact, in addition to the high frequency events, these do not coincide where the vibration maxima are observed and have too small an amplitude to make a significant impact on the overall ppv. The distribution of peak particle velocities does, however, appear to be related to the depth of the pile toe, since the horizontal distance from the pile at which the maxima occur increases as the pile toe depth increases.

Equation 18 and the propagation velocities at the A47 site suggest that surface waves may first appear sufficiently close to the pile to be present on all geophones. However, Nakano (1925) determined that the surface waves do not attain their maximum amplitude close to this limit distance, from which it may be inferred that the surface wave amplitude increases for part of the distance beyond the critical distance derived from Equation 18.

Nakano's analysis was for a subsurface source. The predicted distribution of vibration magnitudes did not require a second source of waves for interference, so the distribution of vibration levels with horizontal distance observed at the A47 site can be explained without the need for constructive interference of vibration arriving from the shaft and the toe.

The occurrence of a peak in the vibration magnitude at some distance from the pile may therefore arise through the processes of Rayleigh wave generation predicted by Nakano (1925). At very close range, of the order of 1m to 3m, there occurred a superposition between the main surface wave and the high frequency event which gave rise to higher levels of vibration in this region. The significance of this superposition diminished rapidly with increasing distance because the high frequency propagated more raidly than the main vibration event. Therefore, the two wave modes only arrived simultaneously close to the pile.

An alternative interpretation of the observed local maxima in the vibration data is not that the maxima represent an anomalously high subset of the data, but that the vibration level closer to the pile is lower than that expected from the more distant data. To investigate this suggestion, Figure 34 presents the components and true resultant ppv for four different pile toe depths at the A47 site, plotted against horizontal distance from the pile. The peak in the ppv-distance data becomes more pronounced as the pile toe depth increases. Figure 35 re-presents the data from Figure 34, from offsets of 9.7m and 23.5m, plotted against the depth of the pile toe.

In Figure 34, when the pile toe was at a depth of 12m the decay is continuous for all components of vibration. As driving progressed, the ppv recorded at the second geophone position (23.5m offset) and beyond increased slightly. A greater change occurred at the closest geophone (9.7m offset) where a decrease in the level of vibration was recorded as the toe depth increased. This observation supports the suggestion that the vibration maximum in the ppv-distance plots occurs not because of a high level of vibration at the second geophone position, but because a low level of vibration occurs close to the pile.

In Figure 34, the resultant peak particle velocity at 9.7m from the pile (Figure 35a) decreased throughout the pile drive. At an offset of 23.5m (Figure 35b), an initial increase of ppv with toe depth occurred, after which the resultant ppv remained approximately constant. Figure 35 demonstrates that the maxima at 23.5m shown in Figure 34 were a result of a low vibration level close to the pile, rather than an anomalously high level of vibration occurring at the greater distance. The horizontal radial component, in particular, dictated the changes in vibration level, decreasing rapidly at a distance of 9.7m as the pile was driven. At 23.5m, the horizontal radial component follows a similar trend to the resultant ppv. This observation concurs with Selby (1991) and Jongmans (1996) who reported that the ppv maxima only occurred in the horizontal radial component. It is therefore not necessary to have constructive superposition of two waves, as proposed elsewhere in the literature, to explain the observed distribution of peak particle velocities.
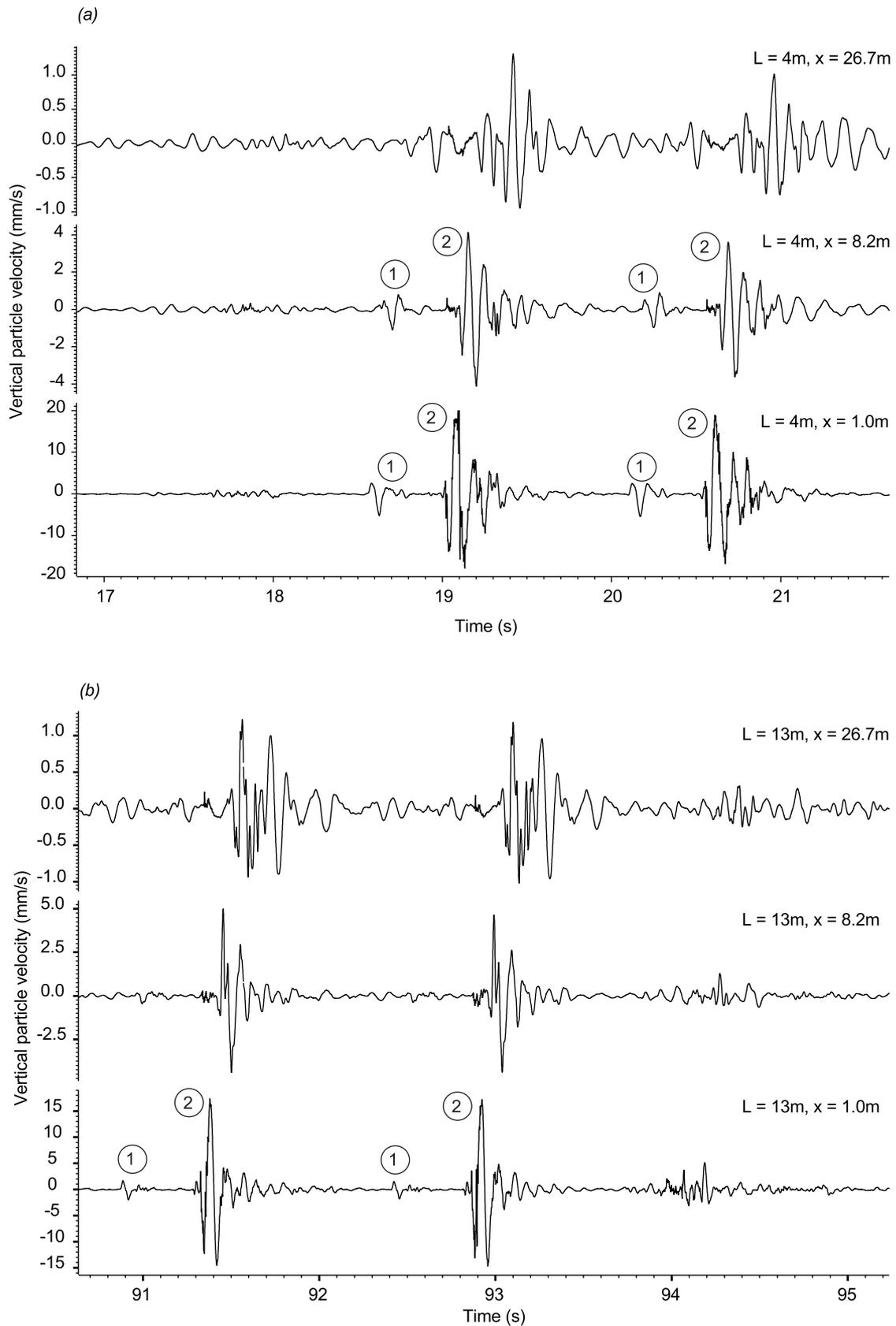
Figure 33 Vertical component of vibration at the A47 site from (a) the first two impacts of driving the lower section of a pile using the maximum nominal energy (toe depth approximately 4m) and (b) the end of driving (toe depth approximately 13m). The traces show a relatively low amplitude event ① arriving approximately 0.5s before the main event ②
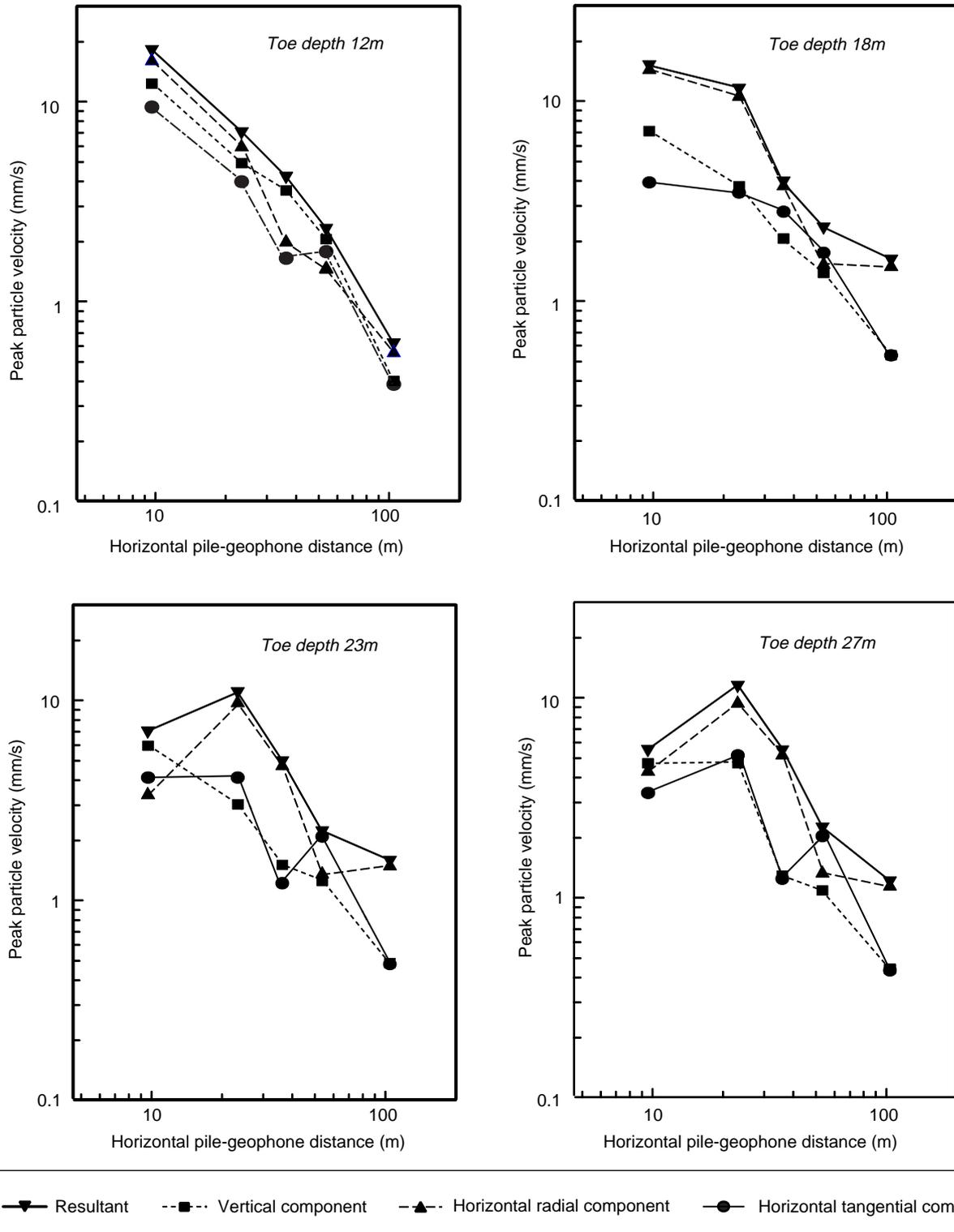
Figure 34 Change in shape of the attenuation curve with pile toe depth (L) when plotted against horizontal distance (d) from the pile during the driving of a pile at the A47 site

Figure 35 Vibration data from Figure 34 for horizontal distances of (a) 9.7m, and (b) 23.5m from the pile

The discussion given above is based largely on the data from the A47 site, from which the largest data set and largest range of horizontal distances and pile toe depths were available. Similar trends were observed from other sites, which demonstrated an increase in vibration levels as piling progressed. In addition, the Dee Crossing site contains data from distances as close as 10m, at which there is an indication of the maxima occurring at approximately 11m fron the pile.
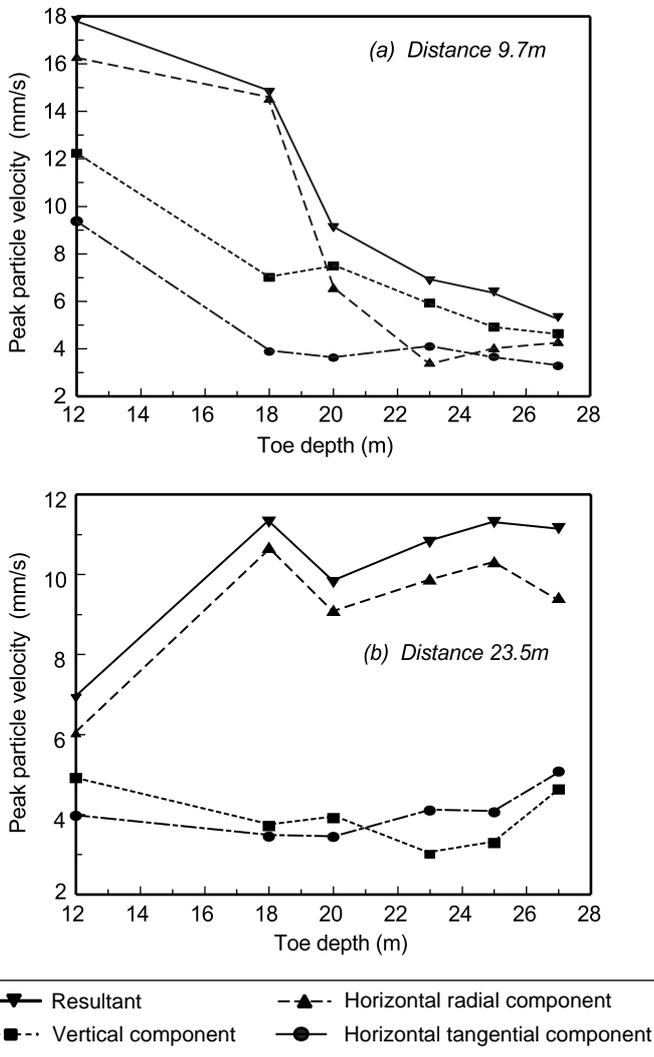
The interpretation of the field data has suggested that the energy transmitted from the pile takes place largely at the pile toe during percussive piling. A relation between the penetration resistance offered by the material into which piles are being driven and the resulting vibration level has been observed, although there are insufficient data available to develop a quantitative relation between these parameters. In most cases, the greatest resistance to driving will be encountered when piles are driven to their maximum depth. The driven depth will therefore be important in determining the vibration level experienced at the ground surface.

5.5 Recommendations for the prediction of vibration from impact piling

The distribution of vibration levels arising at the ground surface from impact piling has been found to be dependent on the penetration resistance of the soil, the depth of the pile toe and the nominal energy input of the piling hammer. Many of the models presented by earlier workers have suggested a square root dependency of ppv on driver energy. However, Ciesielski *et al* (1980) suggested that the vibration level was dependent upon the hammer energy raised to the power of 0.35 and O'Neill (1971) suggested a direct relation between the two parameters. The evidence available from the literature and the current research suggests that the square root energy dependency provides a useable working model, and it is theoretically sustainable, although other factors also need to be considered.

The observation that vibration levels do not decay uniformly with distance may limit the range of applicability of some of the available power law predictors, although they provide a useful estimate of the vibration at distances greater than the embedded length of the pile. The field data show that over distances extending along the ground surface to approximatley the embedded length of the pile, the maximum vibration level experienced during the driving of a pile does not decrease, and is frequently seen to increase, with increasing horizontal distance. The predictive curves from Attewell *et al* (1992a and b) only cover horizontal distances from 2m to 20m, for which smooth decay curves were presented. These curves do not appear to be representative of the observed behaviour over this range of distances, but they do provide a more representative indication of the distribution of the vibration levels at short horizontal distances from the pile than do the simple power law predictors.

The analysis of the data from the A47 site in particular has indicated that the vibration level arising from impact piling is dependent upon the penetration resistance of the ground into which the pile is being driven and on the slope distance from the pile toe to the observation point. In many cases, piles will be driven to refusal where they will generally encounter the highest driving resistance from the ground. The highest levels of vibration arising at any given *slope* distance from the toe will therefore be expected to occur at the end of driving. It follows that the founding depth of the piles will have a significant influence on the vibration levels occurring at the surface, particularly close to the pile.

The various methods for predicting levels of vibration arising from impact piling appear to provide a reasonable, but approximate, basis for preliminary prediction of vibration. Most of the predictors are based on horizontal distance from the pile. This is satisfactory at large distances but it is suggested that, as a rule of thumb, for horizontal distances less than the embedded length of the pile, the horizontal distance is not appropriate.

The guidance given by BS 5228 : Part 4 (BSI, 1992b) recommends that the slope distance to the pile toe is considered. Analysis of the current data indicated that close to the pile, use of the slope distance to the pile toe in the various predictive equations can cause an underestimation of the ppv. The British Standards prediction is for the

vertical component of vibration which, it has been shown above, is often not the largest component. Furthermore, one relation is presented for all ground conditions, whereas the type of ground penetrated has been shown to be important in determining vibration levels.

The recommendations given by Eurocode EC3 Chapter 5 (CEN, 1998) for impact piling therefore appear to be more appropriate, although the guidance is presented in terms of horizontal distances. The quadratic attenuation curves presented by Attewell *et al* (1992) are not wholly representative of the behaviour close to percussively driven piles, but the curvature inherent in these relations reduces the overestimation which arises from the more simple relations when the distance from the pile is specified horizontally.

Based on the new data and the existing predictors, it is suggested that the following relation is used for the prediction of vibration from impact piling:

$$v_{res} \leq k_p \frac{\sqrt{W}}{r^{1.3}} \tag{19}$$

where: v_{res} is the resultant peak particle velocity (mm/s);

$\quad W$ is the hammer energy per blow (J);

$\quad r$ is the distance from the toe of the pile to the point of interest (m).

$\quad k_p$ is an empirical scaling factor.

The parameter k_p should be assigned a value of between 1 and 5, as specified in Table 9. Insufficient new data have been acquired during the current study to enable values of k_p to be determined directly from the data. Consequently, the values of k_p specified in Table 9 for the various ground conditions have been calculated such that the value of v_{res} at a distance (r) of 10m equates to the less conservative of the values for each of the ground classifications given by Whyley and Sarsby (1992) and CEN (1998). An offset of 10m was used as the datum distance for two reasons. Firstly, at distances measured along the ground surface (x) of less than approximately 10m from the pile the relation between v_{res} and x is often different to that at greater distances. Secondly, the CEN document does not state the range of distances from which the data used to derive the predictor. However, consideration of historic data in the literature revealed that most of the field data were determined at distances of less than 20m (Head and

Table 9 Values of k$_p$ for use in Equation 19 for percussive piling in various conditions

| Ground conditions	k_p
All piles driven to refusal.	5
Pile toe being driven through: Very stiff cohesive soils; Dense granular soils; Fill containing obstructions which are large relative to the pile cross section.	3
Pile toe being driven through: Stiff cohesive soils; Medium dense granular soils; Compacted fill.	1.5
Pile toe being driven through: Soft cohesive soils; Loose granular soils; Loose fill. Organic soils.	1

Jardine, 1992). Seventy per cent of the data in the database compiled by Head and Jardine were from within 20m of the pile and had a median distance of 11m. The value of k_p for piles driven to refusal has not been specified elsewhere and has been determined from the new data. A value of k_p of 5 predicts levels of vibration which were exceeded by only a few data points from the data acquired during the current study.

Equation 19 and Table 9 yield vibration levels which are unlikely to be exceeded in most cases. In general, the highest vibration level at any slope distance from the toe arises when the pile is driven to its maximum depth. The value of k_p for use in Equation 10 should therefore be selected for the material encountered when the pile is fully driven and predictions should be based on the corresponding distance. Equation 19 and the values of k_p from Table 10 are compared with the field data from the current study in Figure 36.

Some doubt has been expressed in the literature regarding the relation between the resultant ppv and the nominal driver energy. While it is clear that the vibration level arising from impact piling increases with increasing driving energy, there are insufficient data from hammers with different energies within the new database to verify the relation, so the conventional form of presentation is retained.

Table 10 Details of ground improvement sites

Site	Plant	Geology (0-2m/2-5m/5-10m/10+m)	Topography	Range of distances (m)
A50 - Stoke-on-Trent	Vibro stone cols	Made/made/claystone/claystone&coal	In excavation	10.7 - 45.0
A564 - Derby Southern bypass, Derby Road	Vibro stone cols	Clay/gravel/clay/ mudstone	At grade/flat	6.90 - 109.0
A564 - Derby Southern bypass, Trent Bridge	Vibro stone cols	Sand&clay/clay/silt/mudstone	At grade/flat	98.9 - 114
Coventry business park	Dynamic compaction	Made/mudstone/NA	At grade/flat	5.0 - 119.4
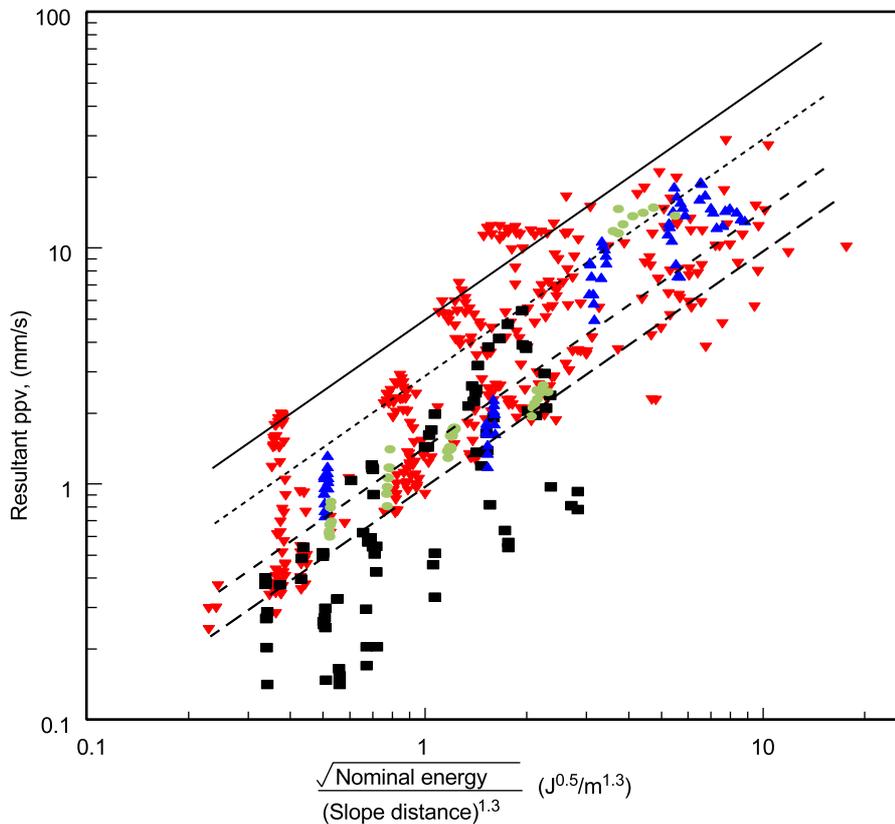
Figure 36 Comparison of field data from percussive piling sites in the current study with predictions calculated using the proposed predicator (Equation 19 and Table 9)

The above expression provides a reasonable estimate of the vibration levels observed on the sites studied within this research. The expression is plotted against the field data in Figure 36. While it is acknowledged that the predictor does not fully describe the detailed distribution of ppv with distance, it does provide a reasonable envelope to the data set.

The research has shown that even the most conservative of the existing predictors can underestimate vibration levels in some cases. While it would be possible to specify a more conservative prediction method, over conservatism could lead to excessive requirements for monitoring or mitigation. It is therefore recommended that the method given above is used for preliminary predictions. On sites where the vibration level is predicted to be close to the prescribed limits, or where particularly sensitive receptors are present, a more detailed assessment should be undertaken. If the prediction remains marginal, then monitoring of the works is recommended to ensure that the limits will not be exceeded, or to develop an operating method which does not exceed the required limits. Such measurements should be undertaken early in the contract and, if possible, at the least sensitive locations on the site.

5.6 Analysis and interpretation of data from vibratory piling

Vibratory pile drivers (vibrodrivers) operate by a different means to impact driving. Vibrodrivers contain one or more pairs of contra rotating eccentrically loaded shafts which generate vibration when set in motion. The driver is clamped to the top of the pile so that oscillation of the eccentrics causes the pile to vibrate. The vibration reduces the shear strength of the ground to the extent that the pile is driven into the ground under the combined weight of the pile and driver. The vibration transmission into the ground is also by different means than that during impact driving, as described in the following sections.

5.6.1 The influence of vibrator energy on vibration level

A number of empirical predictors for vibration levels arising from vibrodrivers have been presented in the same format as those for impact piling (for example Uromeihy, 1990; Attewell *et al*, 1991; BSI, 1992b; CEN, 1998) with the difference that, for vibrodrivers, the energy term is specified as the nominal energy per cycle (Tables 5 and 6). While scaling the distance term according to the nominal hammer energy has been shown to be appropriate for impact piling, the current analysis has indicated that other mechanisms may be more important for vibratory driven piles, as discussed below. Combining the data from all the sites where impact piling was monitored, the scatter was reduced by plotting the data against the distance normalised by the energy, when compared with plotting against distance only. For vibrodrivers, however, scaling the distance according to the driver energy per cycle did not improve the correlation (Figure 37). A similar effect was determined using the data from Uromeihy (1990).

The data from Uromeihy (1990) included many different combinations of plant, pile types and ground conditions. Considering data from only those drivers which had a nominal driving energy of 10.7kJ per cycle, a data set which included 11 sites, at any given distance from the pile, the scatter of the data was similar to that observed for all data from all sites (Figure 38). Clearly there must be some significant influence on the vibration level other than the nominal energy per cycle.
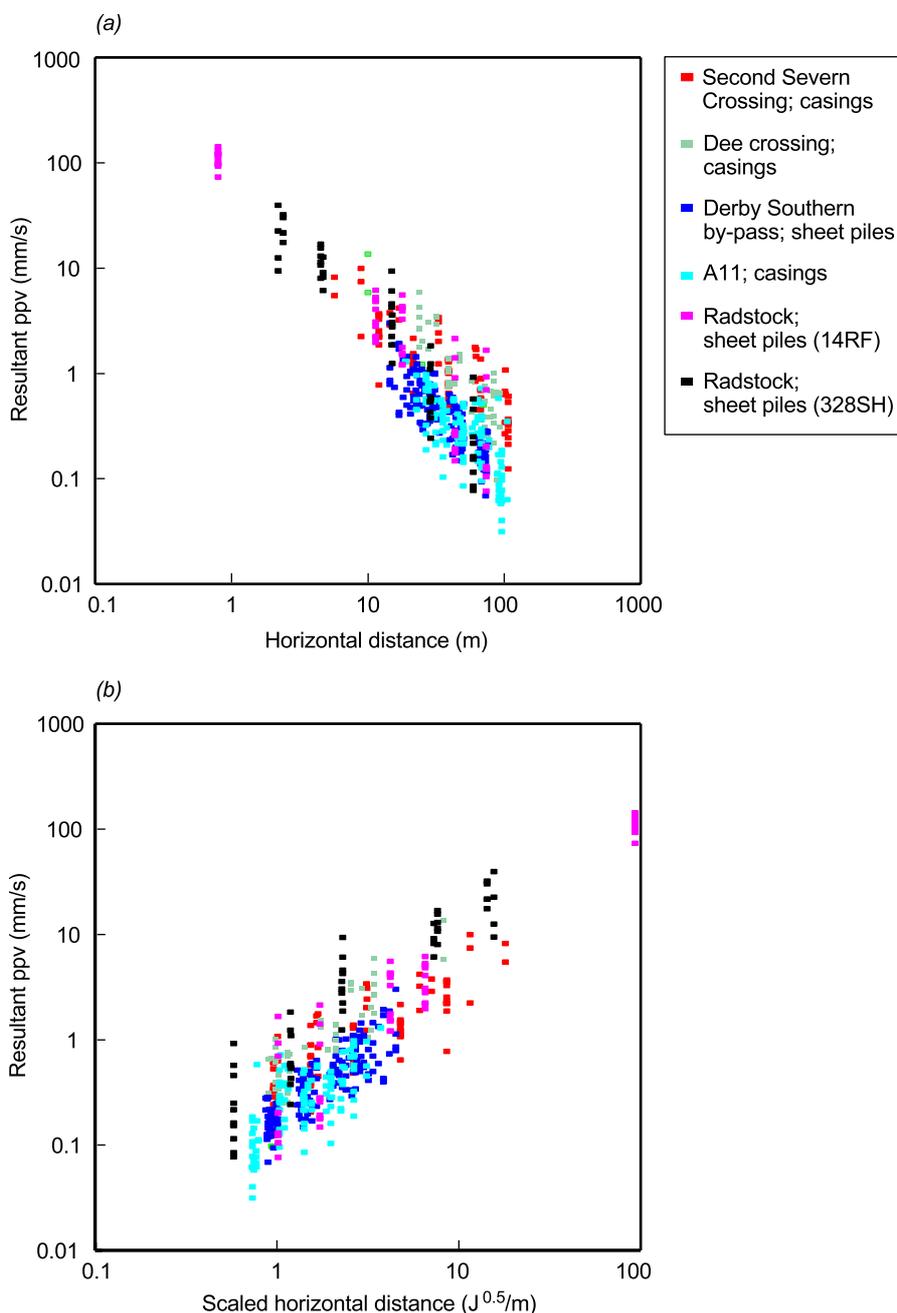

Figure 37 Field data from all vibrodriving sites in the current study plotted against (a) the horizontal distance from the pile and (b) the horizontal distance scaled by the nominal energy per cycle of the vibrodrivers
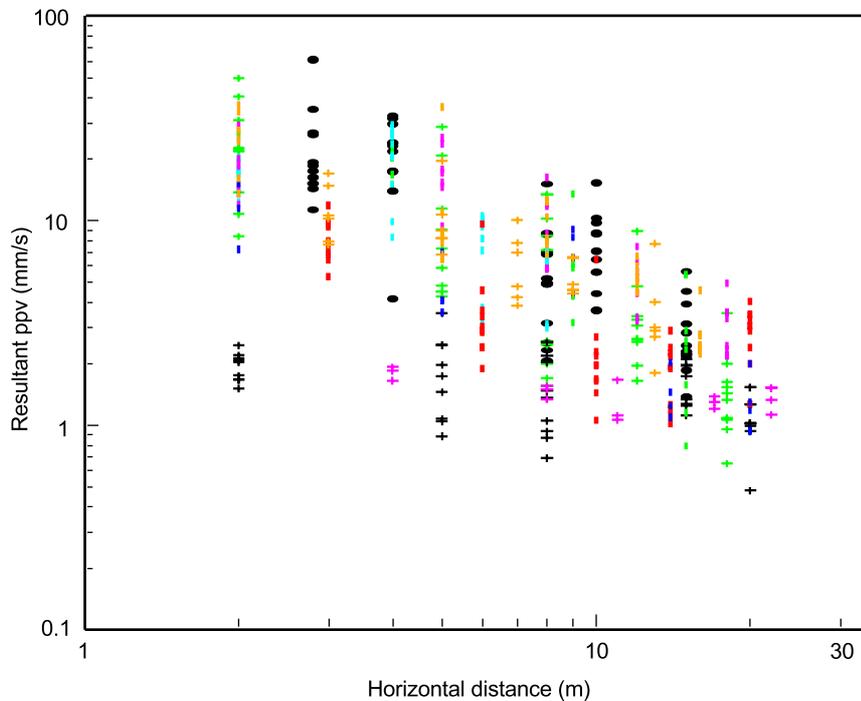
Figure 38 Vibration data for vibrodrivers with a nominal energy of 10.7kJ/cycle. Symbols represent different sites (data taken from Uromeihy, 1990)

Comparison of the vibrations arising from installation of vibratory driven piles with those from pile extraction (Figure 39) showed that, at any distance measured horizontally from the pile, the vibration levels were very similar for both operations on any particular site. Since the maximum vibration levels arising from both vibratory installation and extraction are similar, it appears that the vibration from vibratory piling arises mainly as a result of friction between the shaft and the soil. Although during installation there must be some energy transferred at the pile toe, it is concluded that the energy transfer at the toe has little influence on the level of vibration at the ground surface.

5.6.2 *The influence of pile type and ground conditions on vibration*

There is limited evidence from Uromeihy's data which indicates that the vibration levels may depend on the geometry of the pile. Two of the sites were adjacent, and therefore the geology was approximately the same. Steel casings were driven to 20m at one location while 12m long H piles were driven at the second site. The shorter piles were driven through made ground, alluvial clays and sands, and fluvioglacial sands and gravels. The longer piles subsequently encountered soft clay and silt. Over most of the range of distances for which data were acquired, the casings generated higher levels of vibration than did the H piles, at any given depth. This suggests that the pile type may be important, since the surface area per unit length of pile, and therefore the resistance to driving, was likely to be greater for the casing than for the H pile.

The resistance to driving between the pile and the soil will depend on properties of both the soil and the pile. To investigate the change in vibration level as the pile toe depth increased, an envelope to the resultant ppv was

determined for the entire period of driving a number of piles (Figure 40). This revealed that there was no common pattern of when, during the drive, the maximum vibration level would arise from a particular pile, although on any particular site the change in ppv with time was repeatable. For extraction, the maximum vibration occurred at the start and reduced as the pile was extracted.

In addition to the transient elevated levels of vibration which occur during start up and run down of vibrodrivers (Section 5.6.3), other changes in vibration level occur during the main phase of driving, as illustrated in Figure 40. For all cases, particularly if the pile remained in the ground for a significant length of time, it would be expected that the maximum vibration levels generated during pile extraction would occur at the start of the process and decrease as the length of pile within the ground decreased, since this would be how changes in the frictional resistance would occur. It would also be expected that the vibration levels on extraction would be of a similar magnitude to those arising during driving. These characteristics were exhibited by the field data with the ppv arising during extraction exhibiting a continuous decrease during pile extraction (Figure 40a). Considering the driving data qualitatively, the four sites for which geological data are available are described in the following paragraphs.

The A11 site

The A11 site comprised dense fluvioglacial sands overlying chalk. The chalk was of increasing grade with depth. Figure 40b illustrates the change in vibration level during the driving of the entire length of one casing. Following the start up transient (from zero to 18 seconds), the vibration level was initially relatively low, between 18 and 30 seconds. The vibration level then increased until

55

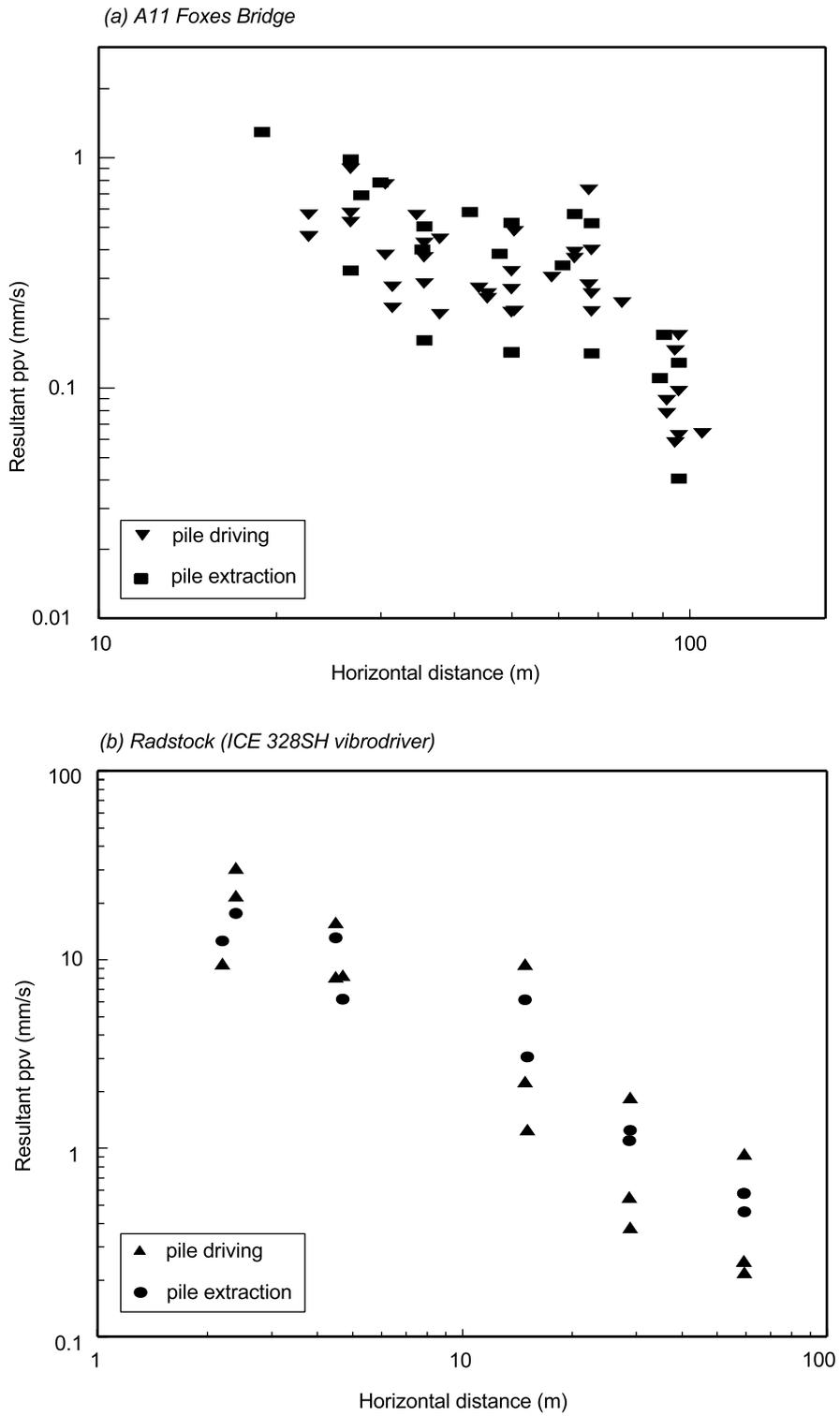**Figure 39** Ground vibration data from the vibratory driving and extraction of piles at (a) the A11 Foxes Bridge site and (b) Radstock, using the ICE 328SH vibrodriver

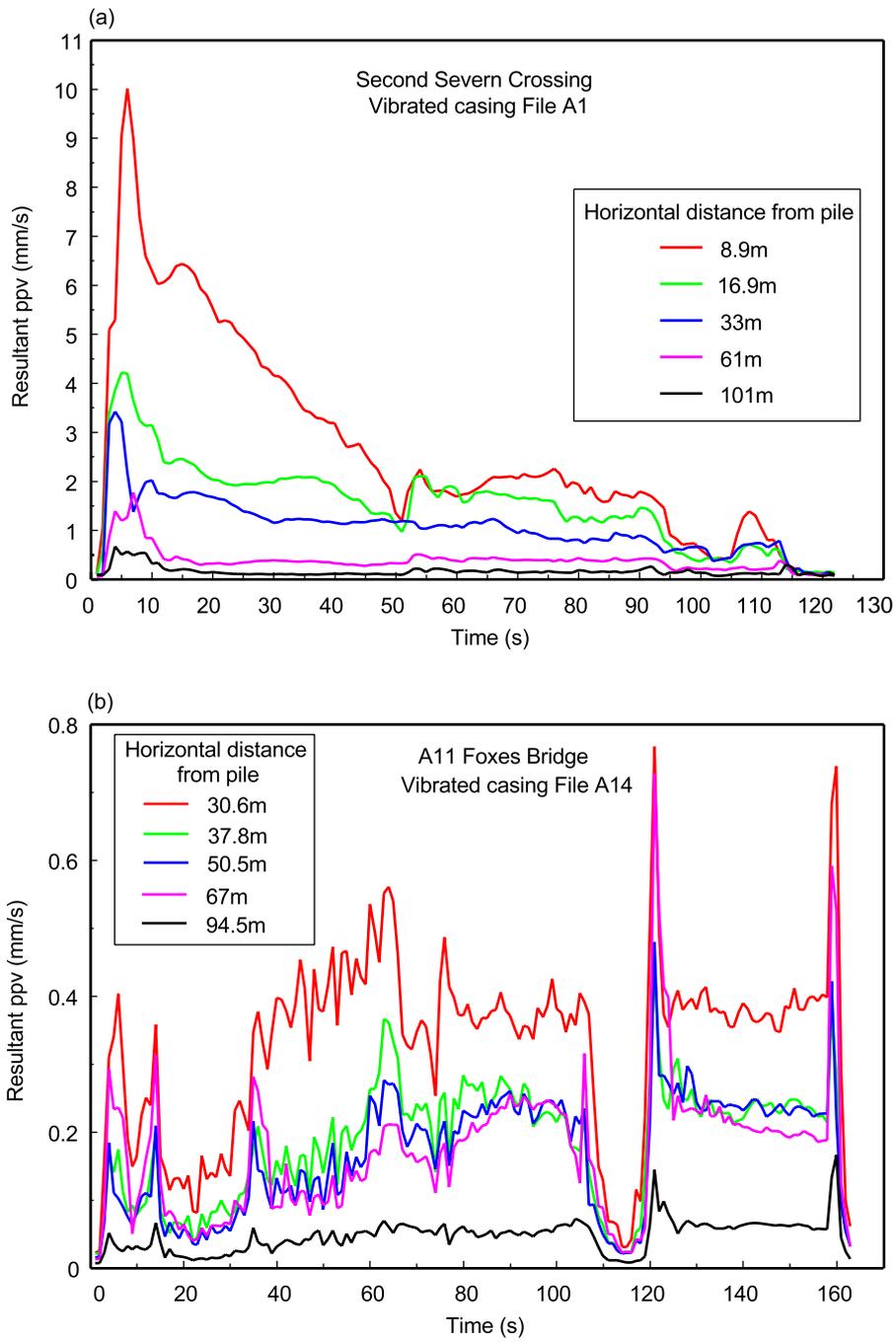**Figure 40** Plots of ppv vs time during (a) pile extraction at the Second Severn Crossing and (b) driving at the A11 site
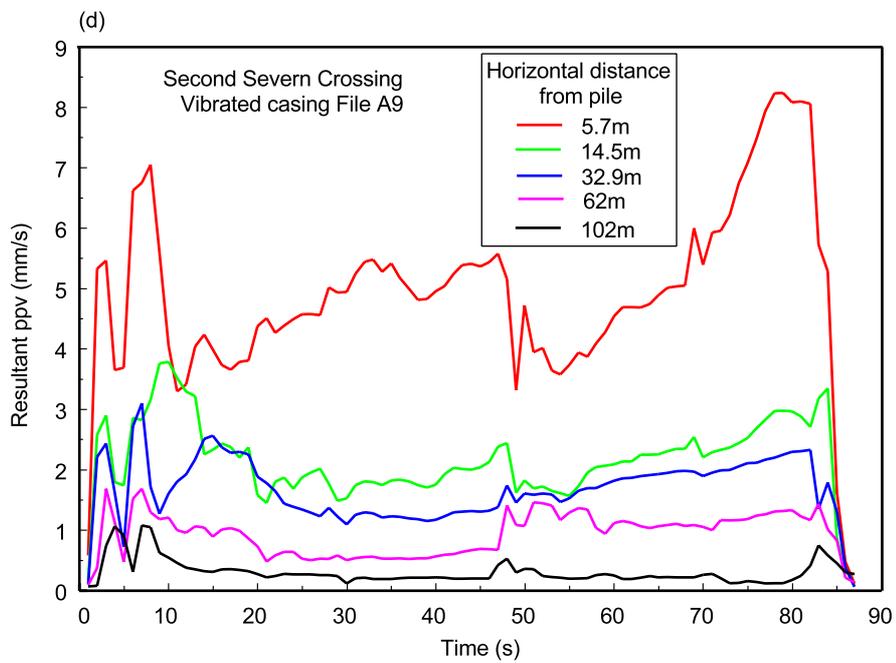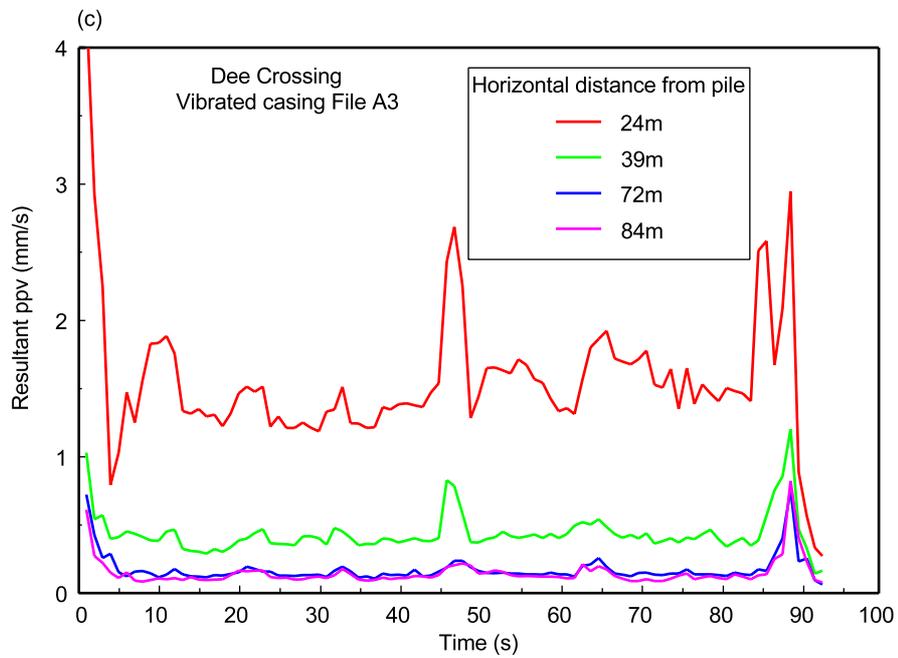
Figure 40 (continued) ppvs measured during vibratory casing driving at (c) the Dee Crossing and (d) the Second Severn Crossing
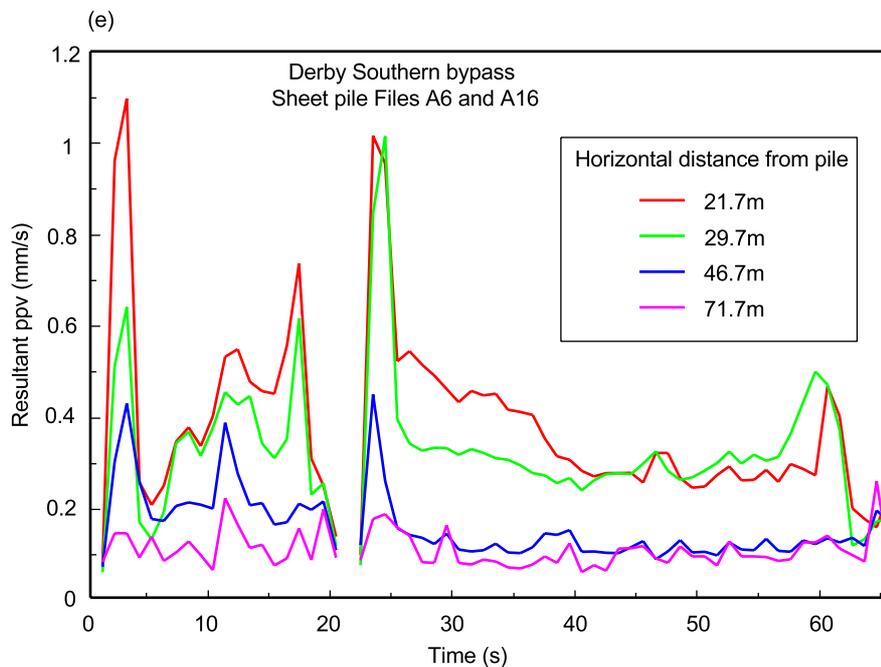
(e)

Figure 40 (continued) ppv from vibratory driving of sheet piles at the Derby Southern Bypass

about 60 seconds from the start and then remained approximately constant throughout the rest of the drive, except when the driver was briefly stopped and restarted. The initial low levels of vibration at the start of driving, occurred while the pile was penetrating the sand. The beginning of the increase in the vibration level may indicate the transition from the sand to the chalk. The chalk at depth may have sufficient strength to be self supporting, so that the interaction between the pile shaft and the chalk may initially increase and then remain approximately constant.

The Dee Crossing site
Vibration from driving a pile at the Dee Crossing site are presented in Figure 40c. Only a small general increase in the ppv occurred as the pile was driven, with the exception of the starting and stopping transients, which gave rise to the highest vibration levels. There was also a peak at about 45s, the cause of which is not known. Most of the drive was through clays, which became progressively stiffer with depth. The final 4m were driven within mudstone.

   A possible explanation for there only being a slight increase in the ppv recorded at the Dee Crossing site throughout the drive is that remoulding of the clay by the continuous shearing action of the pile may have caused each incremental length of the contact between the clay and the pile to reach its residual shear strength. Once the residual shear strength is reached, no further change in the contribution to the total resistance made by each incremental length would occur. Therefore, an initial increase in resistance would occur as the weaker materials

were penetrated, but the resistance would then increase only slightly as the total length of remoulded soil in contact with the pile increased.

The Second Severn Crossing site
Figure 40d shows the vibration levels from driving a pile casing at the Second Severn Crossing site. The pile was sleeved to a depth of 6m, so all the vibration data are for driving through soft to very soft silty clays and into the firm to stiff red marl in which the piles were founded. The vibration record shows an initial high level due to the starting transient. Following the transient period the vibration level generally increased for the remainder of the drive. This may reflect an increase in the resistance to driving as the length of the pile in the ground increased. After approximately 50 seconds, there is a sharp decrease in vibration recorded by the two closest geophones whereas at greater distance, the vibration level continued to increase. It is not clear why this arose, but it serves to illustrate the difficulty in predicting vibration levels arising from vibrodriving.

The Derby Southern Bypass
The piles at the Derby Southern Bypass site were driven in two stages. The first 2.5m were driven before the guide frame was removed, after which the drive was completed. There was therefore a period of 30 minutes between the two driving stages. Figure 40e illustrates the vibration for the whole drive of one pile, with the break in the record indicating the gap between the two stages of driving. The vibration level initially increased and then decreased,

thereafter remaining essentially constant for the rest of the driving period. If the transients periods caused by the shut down and restart are disregarded, the vibration levels were similar before and after the interruption to driving. The geological profile at the Derby site comprised sands and clays to a depth of 3m, followed by 5m of gravels over firm to stiff silty clays of the Mercia Mudstone series. Three different trends in the vibration level occurred: increase, before the interruption to driving; decrease, for approximately 15 seconds after the restart; and a period where the vibration level remained approximately constant. These changes in the vibration may reflect the geological changes, but without any record of the toe depth this cannot be verified.

In conclusion, the vibration levels arising from vibratory pile driving appear to be related to the energy transferred to the ground through resistance to movement at the pile-soil interface. The resistance would be dependent upon the ground conditions, the dimensions of the pile and the amplitude of vibration of the driver. However, further work would be required to develop a quantitative predictive model.

5.6.3 Transient vibrations during start up and run down

Vibratory pile drivers operate by the rotation of one or more pairs of eccentric contra rotating weights which are arranged such that the forces generated are vertically polarised. During the start up and run down phases of operation, the rotational frequency increases up to the operating frequency, during which time a resonance phase occurs. This commonly gives rise to vibration levels which exceed those caused by steady state operation. The plots of ppv against time (Figures 40) illustrate the importance of the transient vibration levels which arise during start up and run down of vibratory pile drivers. These transient vibrations are, in nearly all cases, the highest levels of groundborne vibrations experienced during the vibratory driving of a pile and therefore must be considered in predictions.

The measurements made at the Radstock site presented an opportunity to measure vibration from a vibrodriver which could operate in such a way as to avoid the resonances. The vibrodriver studied was an ICE 14RF; similar systems are also available from other manufacturers. The high amplitude transient vibrations are avoided by a system which allows the rotating system to be accelerated up to its operating rotational frequency with the rotating masses balanced. When the operating speed is reached, the masses are rearranged to be out of phase, thereby generating the required vibration. The driver can also be operated conventionally, in which case the high amplitude vibrations will occur.

The vibration level measured close to the pile during driving with the 14RF was similar for both the resonance free and conventional modes of operation. At greater distances the resonance effect was more significant, reflecting the different response of the ground to the changing frequency content (Figure 41). When considering the potential for intrusion, this effect is clearly important, and any measurements made either for prediction or resolution of complaints must capture these transient events and record vibrations over an appropriate range of distances.

5.6.4 Summary and recommended prediction method

Predictions of vibration in the literature are based on the energy per cycle of the vibrodriver, although there are no field data in the literature which are offered in support of such a relation. During the current study, there has been found to be no correlation between the nominal energy per cycle of vibrodrivers and the resulting ppv.

The highest levels of vibration arise during the transient phases of operation at the start up and run down of the vibrator. The vibration during these periods of operation assumes increasing significance with increasing distance from the pile and therefore must be included in a vibration prediction. For use in sensitive locations, vibrodrivers are available which operate without the transient phases and are therefore less intrusive.

The distribution of field data from distances ranging from 1m to in excess of 100m was found to plot as a straight line on logarithmic axes of ppv against horizontal distance from the pile. The distance from the pile for use in prediction of vibration from vibrodriving may be specified as that measured along the ground surface. This is in contrast to impact piling, where it was found necessary to specify distances from the pile toe to the point of interest.

Based on the data acquired during the current study and those from Uromeihy (1990), a total of 1281 observations (Figure 42), a first approximation of vibration which may arise from vibrodriving may be obtained from the expression:

$$v_{res} = \frac{k_v}{x^\delta} \tag{20}$$

where: k_v = 60, with a 50 per cent probability of the vibration level being exceeded;

k_v = 126, with a 33 per cent probability of the vibration level being exceeded;

k_v = 266, with a 5 per cent probability of the vibration level being exceeded;

x is the distance measured along the ground surface from the pile (m); and

v_{res} is the resultant peak particle velocity (mm/s)

δ = 1.2 (start-up and run-down); 1.4 (steady-state); or 1.3 (all operations)

6 Groundborne vibration from ground improvement methods

6.1 Ground improvement techniques

Ground improvement methods are used where the bearing capacity of *in situ* materials must be increased in order that settlements of the completed works are within specified limits. In many cases, these techniques are more cost effective and environmentally acceptable than removing the poor ground and replacement with imported fill. However, significant levels of groundborne vibration may arise from such methods, which may detract from their acceptability. Examples of techniques which have been used are vibro replacement, vibro compaction and dynamic compaction. A detailed review of many of these techniques was presented by Greenwood and Kirsch (1984) and Mitchell (1994), some of which are outlined below.
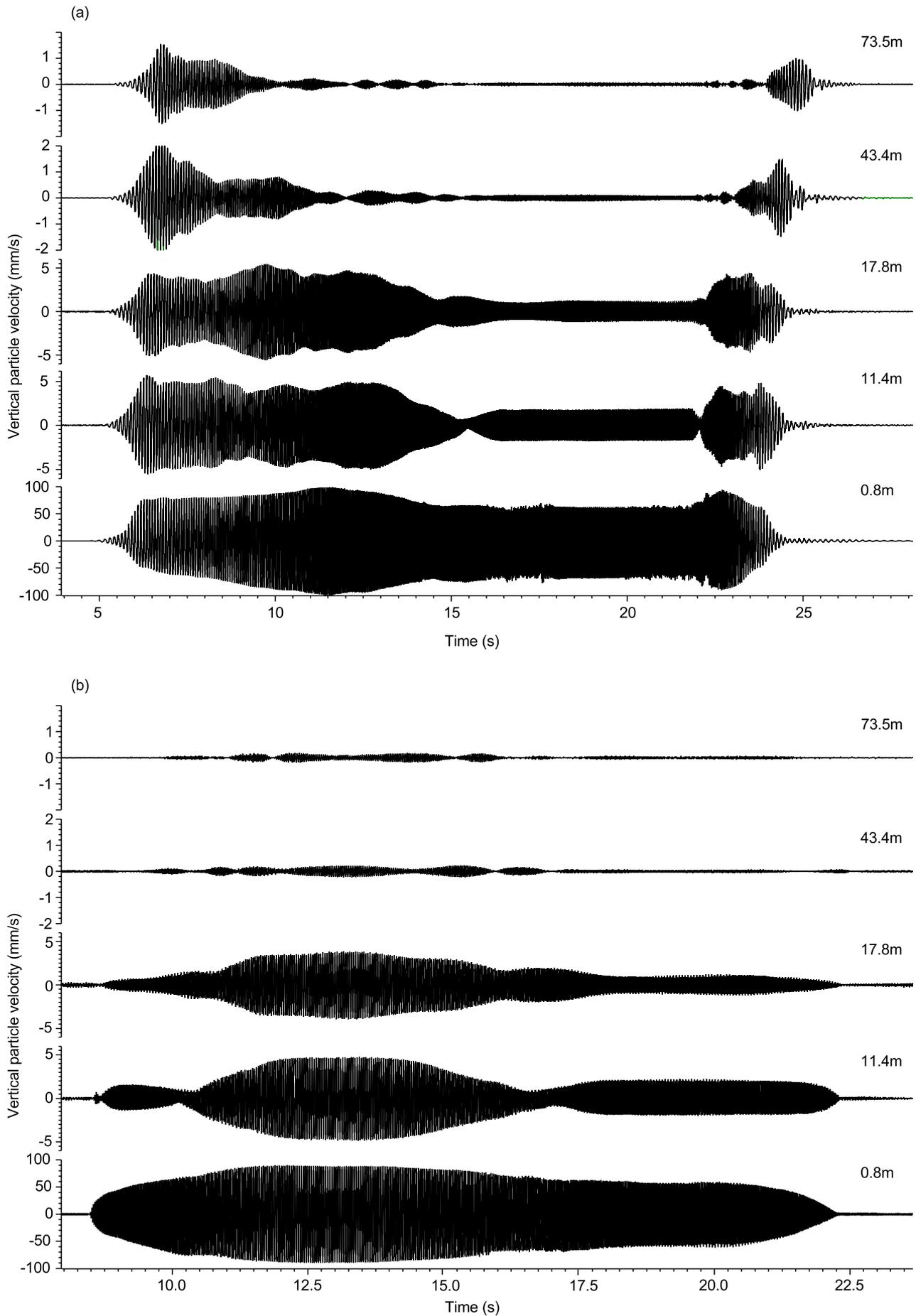
Figure 41 Comparison of vibration arising at various distances from 14RF vibrodriver at Radstock operated (a) conventionally and (b) in resonance-free mode
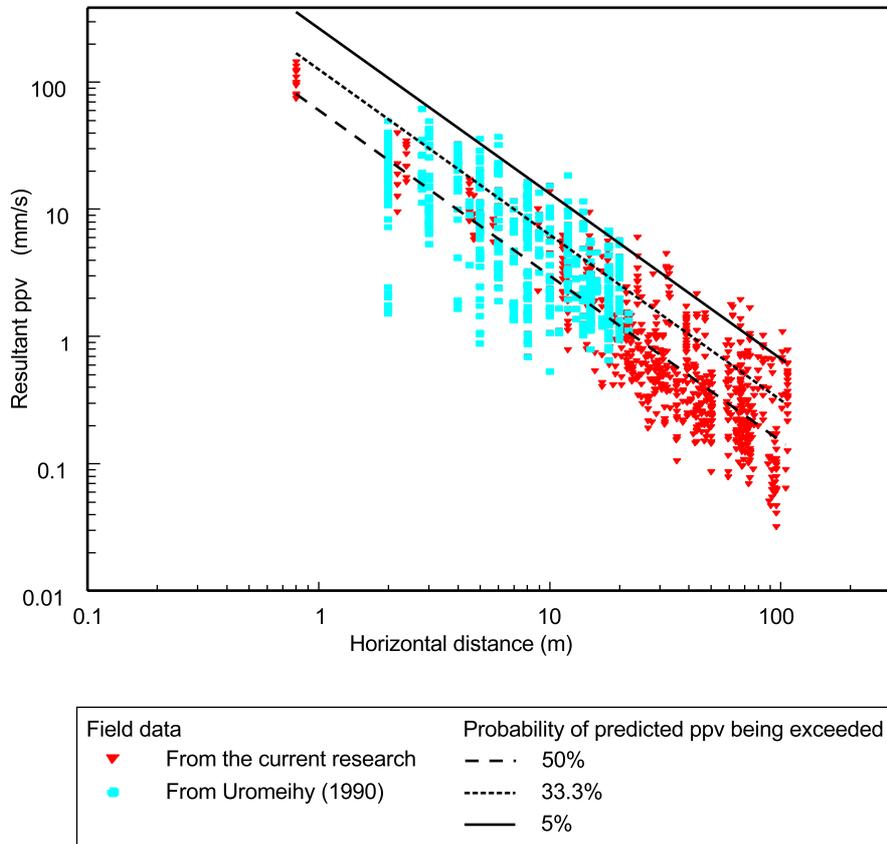
Figure 42 Data from vibrodriving from the current study and from Uromeihy (1990)

6.1.1 Vibratory techniques

A number of different techniques exists which use vibrating pokers to improve the ground conditions which are often referred to globally as vibro flotation. One technique, appropriate for cohesionless soils only, is vibro compaction which operates by causing a rearrangement of the particles into a denser configuration under the influence of vibration.

Vibro replacement (also called vibro displacement) is used to improve cohesive materials, which cannot be treated by vibro compaction. The vibrator is allowed to penetrate the soil to the design depth and the resulting cavity is backfilled with stone. The stone is added in stages after which the vibrator is used to compact the stone to ensure the required bearing capacity and interaction with the ground are achieved. Different techniques are available appropriate to different soil and ground water conditions and depending upon the depth of stone column required.

Greenwood and Kirsch (1984) presented data showing the levels of resultant ppv as a function of scaled distance for four sites on different soils. The data show that ground vibration levels are similar to those generated by vibratory pile driving. The ground conditions appear to have little effect on the vibration level generated by these activities.

6.1.2 Dynamic compaction

Dynamic compaction is a technique whereby a heavy weight is repeatedly dropped on to the surface of the soil to compact the upper layers. Typically the tamper weighs

between 5 and 20 tonnes and is dropped from a height of up to 25m. However, Menard (1974) reported that 170 tonne weights dropped from 40m have been used. A review of historical use, development and the principles of dynamic compaction has been given by Slocombe (1993).

A variation on this technique, a high speed dynamic compaction apparatus, has been reported by Neilson *et al* (1998). This is a self-propelled machine, originally designed for military use for the reinstatement of bomb damaged runways. Compaction is effected by a 7 tonne weight dropped from a height of up to 1.2m onto a metal plate, which remains in contact with the ground.

6.2 Prediction of vibration

Clearly the techniques described above may result in significant levels of groundborne vibration. Mayne *et al* (1984) presented data from a number of dynamic compaction case histories and suggested a predictor which could be used to determine a conservative upper limit of vibration. The predictor was subsequently revised (Mayne, 1985) and presented as:

$$v_{\text{res}} \leq 92 \left[\frac{\sqrt{MH}}{x} \right]^{1.7} \tag{21}$$

where v_{res} is the peak particle velocity (mm/s);
M is the tamper weight (tonnes);
H is the drop height (m);
x is the distance from impact (m).

This expression is also a reasonable, though conservative, upper bound to the data presented by Greenwood and Kirsch (1984).

During the current research, data have been acquired from ground improvement techniques on four sites. Dynamic compaction was in use on one site to improve the properties of made ground for redevelopment. A weight of 8.5t was dropped from heights of 10m and 12m. Vibration measurements made at distances ranging from 5m to 120m verified the predictor proposed by Mayne (1985) which provided a reasonable upper bound to the data (Figure 43). However, the attenuation rate on the site monitored by TRL was lower than that proposed by Mayne, which resulted in an over prediction close to the source and an underestimate of vibration levels at distances in excess of 60m from the impact. In practice, potentially adverse effects other than vibration may need to be considered close to dynamic compaction works. For example, Greenwood and Kirsch recommend that dynamic compaction is not used closer than 20 to 30m from surrounding structures both for protection from vibration effects and to prevent risk from flying debris. Lukas (1986) pointed out that permanent lateral ground displacements may be problematic.

Allen (1996) measured the ground vibration arising from the high speed dynamic compactor and presented a series of site specific power laws for various drop heights. The data are presented graphically in Figure 44 and compared with the vibration magnitudes predicted by Mayne (1985) for a drop height of 1.2m. The predictor appears to be unreliable for this compaction process, under estimating the vibration level by up to an order of magnitude. It is possible that Mayne's predictor is less reliable for low energy dynamic compaction than for more energetic compaction. Most of the data from which Mayne's predictor was derived were from higher energy compaction than the energy of the high speed dynamic compaction apparatus. Furthermore, inspection of Mayne's field data shows that the predictor becomes less conservative as the potential energy of the raised tamper decreases.

No prediction method for ground vibration from other vibratory ground improvement techniques has been found in the literature. Within the current study, vibration from vibro replacement operations was recorded at three locations. On two sites on the Derby Southern Bypass a *Bauer HBM4* bottom feed vibro displacement rig was used. On the A50 at Stoke-on-Trent a *Pennine 300/150 Vibroflot* was used. Details of these sites are presented in Table 10. The data are plotted as resultant ppv against distance in Figure 45 so that they may be used to provide an initial estimate of possible vibration levels arising from these activities on other sites. The equation of the prediction lines presented on Figure 45 is:

$$v_{res} = \frac{k_c}{x^{1.4}} \tag{22}$$

where: k_c = 33, with a 50 per cent probability of the vibration level being exceeded;

k_c = 44, with a 33 per cent probability of the vibration level being exceeded;

k_c = 95, with a 5 per cent probability of the vibration level being exceeded;

x is the distance measured along the ground surface from the pile (m); and

v_{res} is the resultant peak particle velocity (mm/s).

The three lines presented on Figure 45 illustrate the regression line and the predicted levels of vibration which might be exceeded in 33 per cent and 5 per cent of cases, based on the available data. There are insufficient data available to be able to establish a more refined prediction method.

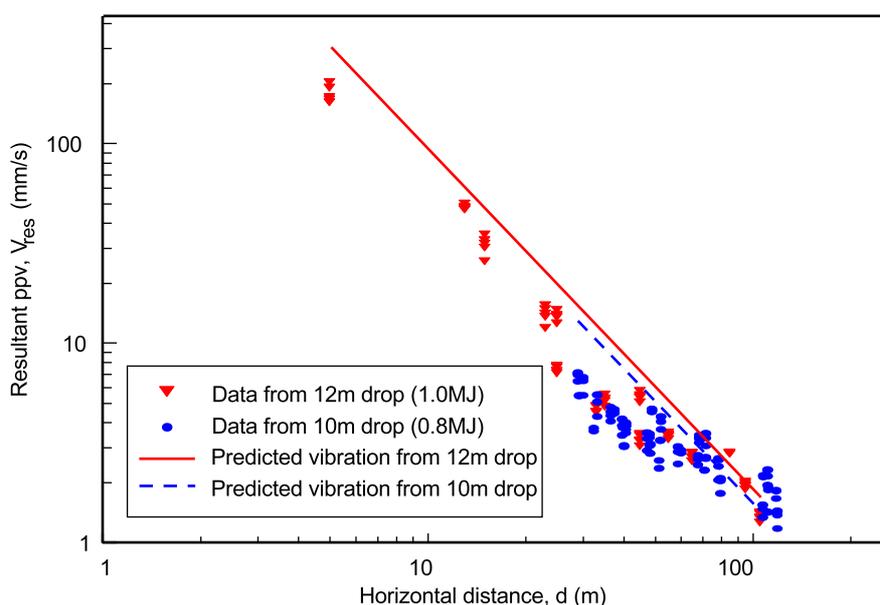**Figure 43** Data from dynamic compaction using an 8.5t tamper at the Coventry site compared with vibration predicted using Mayne's equation (1985)
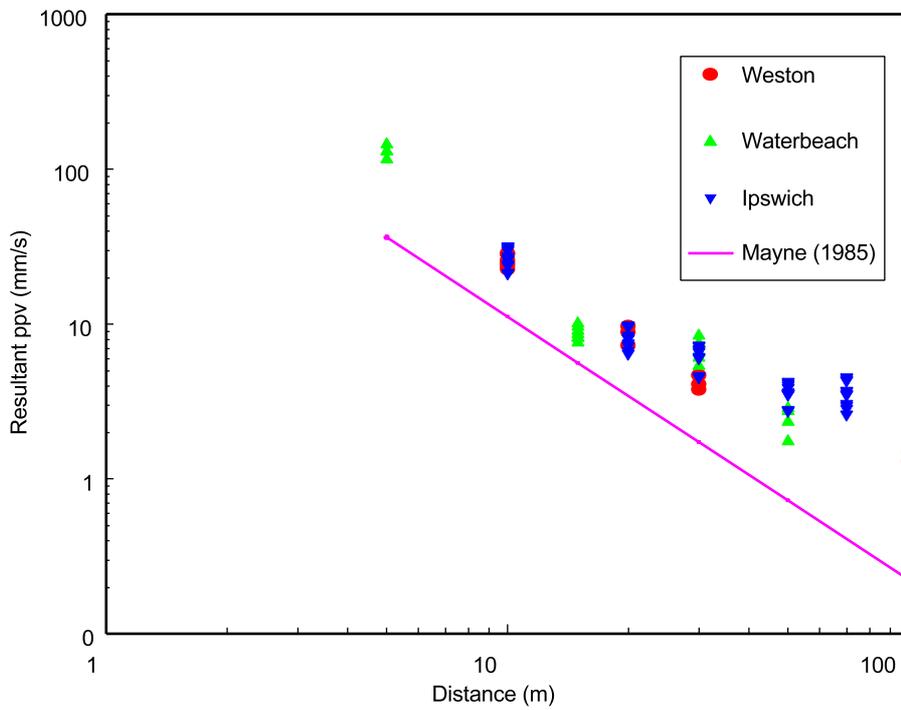
Figure 44 Vibration data from the high speed dynamic compactor reported by Allen (1996) compared with ppv predicted for the 7t tamper dropped from 1.2m using Mayne's equation (1985)
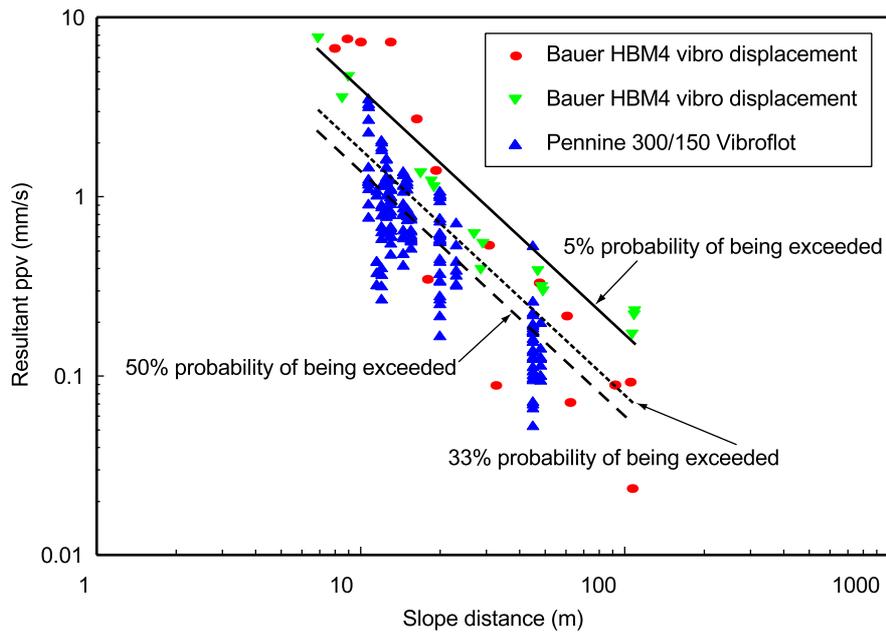

Figure 45 Vibration caused by vibratory ground treatment and the proposed prediction limits

7 Groundborne vibration from mechanised tunnelling

All methods of tunnel excavation involve the dissipation of energy into the ground, which gives rise to groundborne vibration. The highest levels of vibration are generated by blasting and considerable data are available to assist in the prediction of the effects of drill and blast operations in rock tunnels (New, 1986). However, significant levels of vibration may also arise from mechanical tunnelling processes when tunnel boring machines (TBMs) or other excavators impact the tunnel face. Additionally, vibration may occur due to processes such as the percussive driving of spiles and dowels. Such vibration may differ in both duration and frequency content from that due to excavation activities. In addition to considering potential damage and perceptible vibration, for underground construction it is also necessary to establish the potential for disturbance by groundborne noise.

7.1 Previous studies

When compared with many other civil engineering activities, there have been relatively few reports of the levels of groundborne vibration generated by mechanised tunnelling works. Verspohl (1995) commented that 'there is hardly any literature...of vibrations caused by tunnelling'. New (1982) reported three case studies, which were reproduced by Flanagan (1993) who added data from two American tunnelling sites. Fornaro *et al* (1994) presented data from a 3.9m diameter TBM and a high energy hydraulic hammer used on the same sedimentary rock site. The latter authors acknowledged that since their equipment could only record for periods of a maximum of 10 seconds duration, it was not entirely appropriate for measuring vibrations from TBMs. Their data were presented using the KB system recommended for the assessment of human perception by the German vibration standard DIN 4150 Part 2. This approach complicates comparison of the data with the peak particle velocity (ppv) data format more commonly used in the UK because the KB calculation contains a frequency value. However, the KB values presented by Fornaro *et al* (1994) have been included here by conversion to ppv based on the frequency of the maximum amplitude shown in the spectra presented in their paper. The data for the hydraulic hammer have also been represented in terms of ppv by Godio *et al* (1992).

For sources of vibration which are below the ground, such as underground railways and tunnelling works, vibration disturbance may arise through the generation of groundborne noise. This is audible noise which occurs within buildings when vibration transmitted into the building causes the oscillation of floors, ceilings or walls which then radiate sound. Analysis of each signal to determine the root mean square (rms) vibration level in octave frequency bands enables the potential groundborne noise to be predicted. This calculation uses the empirical relation proposed by Kurzweil (1979) and was validated for domestic properties above London Underground trains in tunnels using data acquired by TRL (Greer, 1993):

$$L_p = 20 \log_{10} v_{rms} + 93 \qquad (23)$$

where L_p is the resulting octave band sound pressure level within the room (dB);

v_{rms} is the rms vertical particle velocity (mm/s) measured in the free-field.

Sound pressure levels determined by this expression are evaluated in decibels (dB) referenced to the standard pressure of 2×10^{-5} Pa. Spectral analysis of the vibrations enables A-weighting corrections to be applied so that human response can be fully assessed.

7.2 Vibration data

Vibration data acquired by TRL from mechanised tunnelling are summarised in a series of graphs presented in Figures 46 to 49. In each case, excavation operations have been plotted as solid lines, other tunnel construction operations as broken lines, and a selection of other vibration sources (which are presented for comparison) as dotted lines. In addition to the new data, other data from the earlier literature have also been included. Further details of the measurements are presented in Table 11.

Figure 46a plots regression lines and 46b upper bound lines for resultant ppv against distance. Comparison of the regression and upper bound lines gives an indication of the scatter in the data. The data are presented in terms of true resultant data, except for the TBM data given by Nelson *et al* (1984) and Fornaro *et al* (1994) which are component values, and the data from Cardiff, Warrington and Sutton, which are pseudo resultant ppv. Analysis of the new data showed that, on average, the maximum component ppv value was ten per cent smaller than the true resultant. Therefore, given the scatter in such data, the maximum component could be used to obtain a reasonable but slightly low estimate.

Spectral analysis of the data revealed that energy from excavation sequences typically occurred within a bandwidth of 10 to 100Hz, although New (1978) observed higher frequencies closer to the tunnelling machine at Warrington. Dowel installation also generated vibration at higher frequencies, reaching 300Hz at a distance of 20m from the source at Round Hill.

In order to obtain an indication of the potential for vibration disturbance, 16 hour vibration dose values were calculated for the three chalk sites. These are presented as a scatter plot in Figure 47. The values were calculated by assuming that the rmq particle velocities occurring for the duration of each recorded signal existed for the whole 16 hour period.

Figure 48 presents the predicted levels of groundborne noise from the sites where the data were of sufficient magnitude to enable meaningful values to be calculated. As for the ppv data, these data are also presented as regression lines and upper bound lines.

7.3 Potential for damage and disturbance by tunnelling vibration

Figure 46 clearly demonstrates that blasting works for the Frome Valley sewer generated considerably higher levels
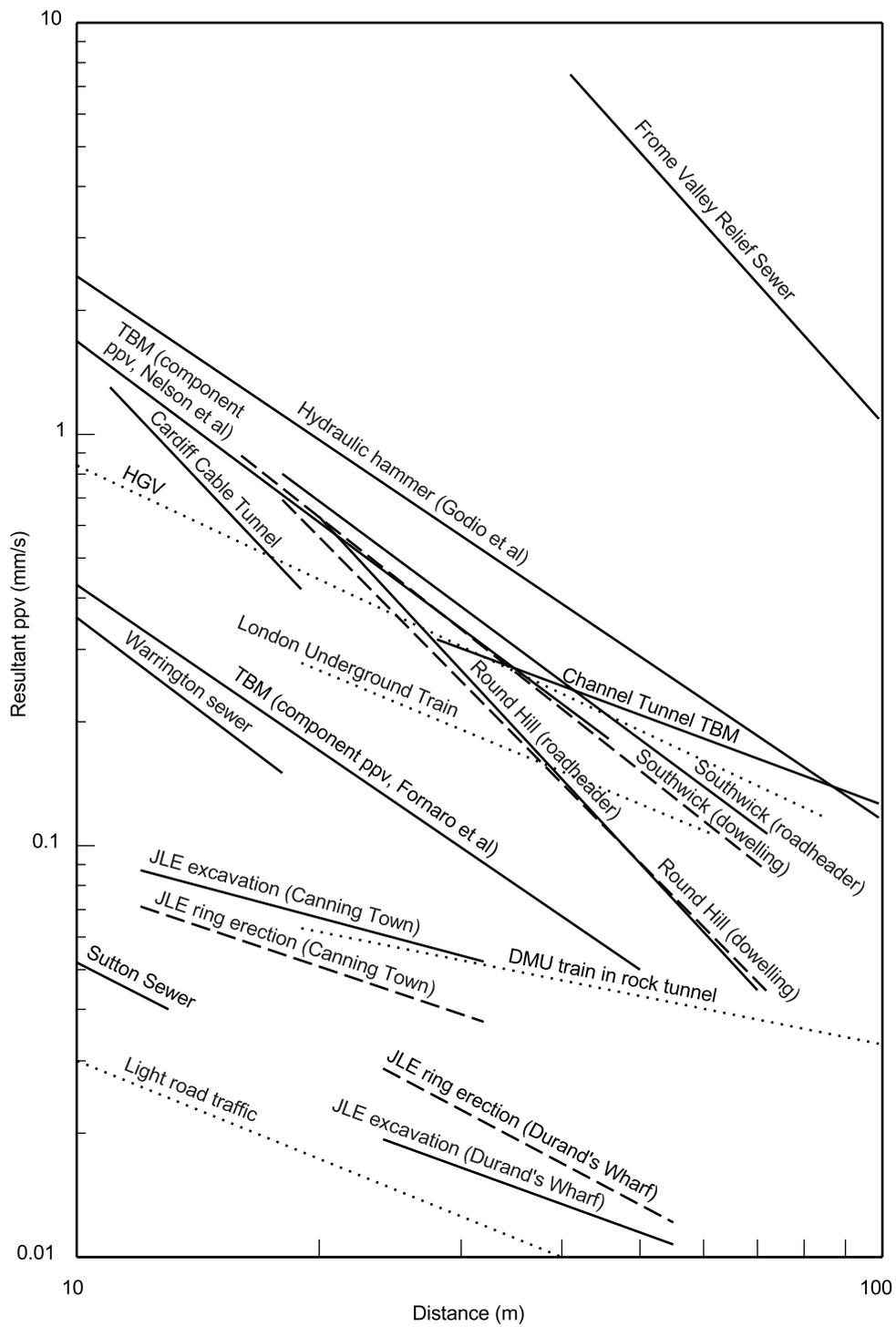
Figure 46a Regression lines for blasting data from the Frome Valley sewer compared with those for various tunnelling operations
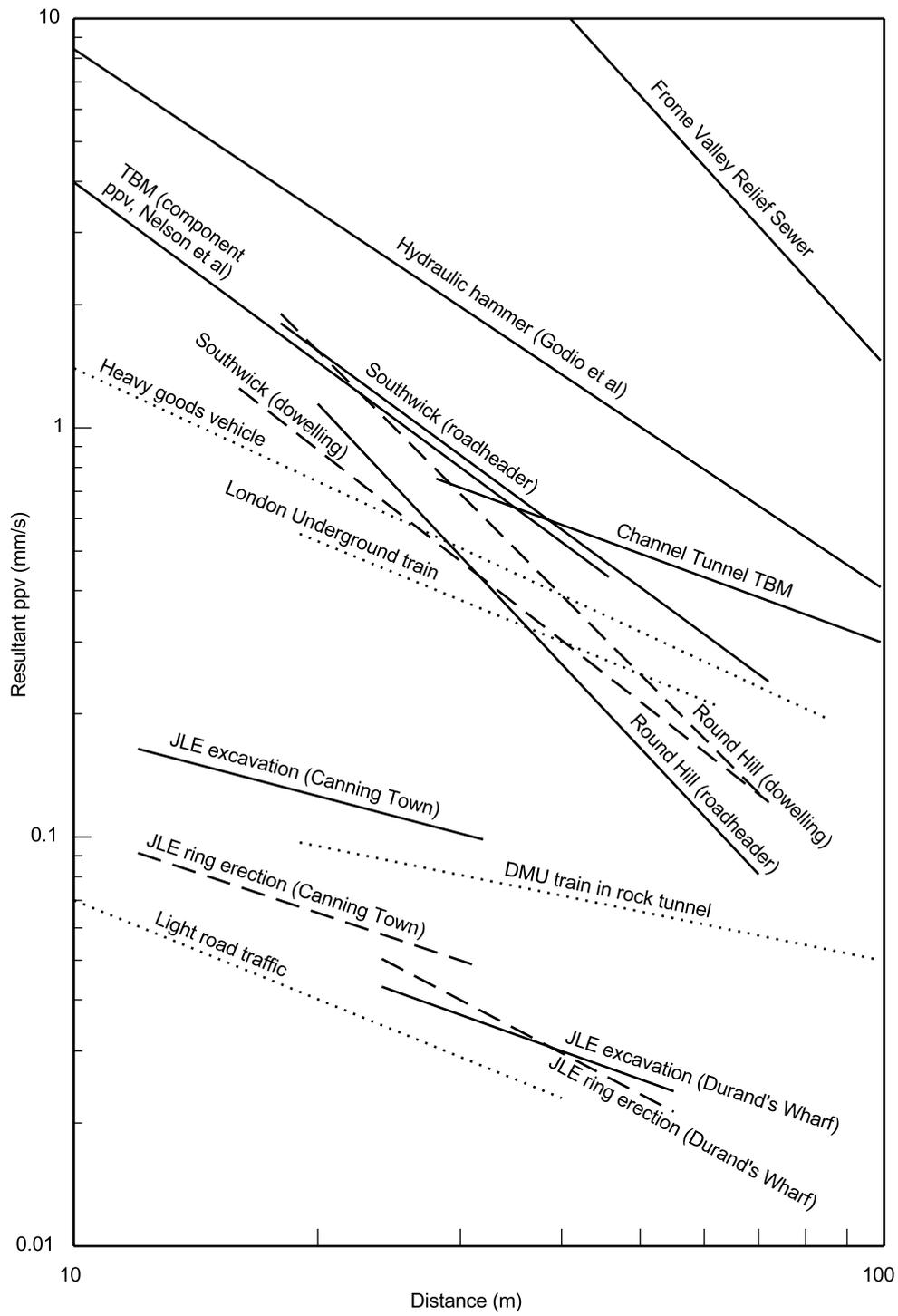
Figure 46b Upper-bound lines for blasting data from the Frome Valley sewer compared with those for various operations
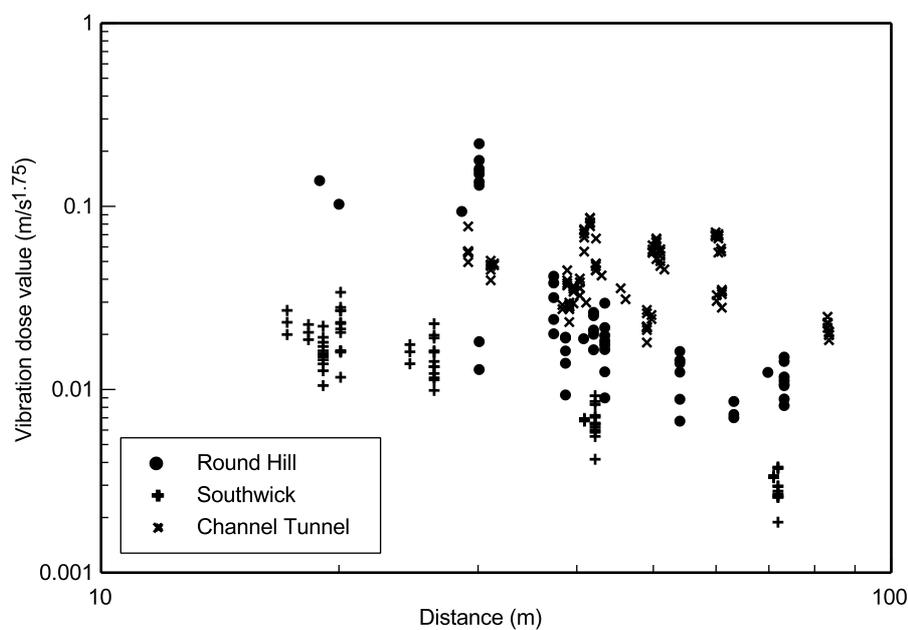
Figure 47 Calculated vibration dose values for tunnelling operations

Table 11 Summary of tunnel vibration data sites, including the drill and blast sewer used for comparison

Site	Project	Geology	Excavation method	Tunnel diameter (m)	Tunnel cover (m)
Canning Town	Rail (JLE Contract 110)	London Clay	Lovat EPB shield	5.14	10.5
Durand's Wharf	Rail (JLE Contract 107)	Woolwich and Reading Beds (sands and clays)	Herrenknecht Mixshield	5.13	21.2
Round Hill	Road (A20)	Lower Chalk	NATM Voest-Alpine ATM-70 roadheader	10.5 (ht)	17.5 - 20
Holywell Coombe	Rail (Channel Tunnel)	Gault Clay/ Lower Chalk	Howden full face TBM	8.72	22.6 - 55
Southwick	Road (A27)	Lower Chalk	NATM Voest-Alpine ATM-70 roadheader	10.5 (ht)	12
Warrington[†]	Sewer	Cohesionless drift with cobbles and some sandstone	Bentonite shield; full face TBM; disc cutters	2.44	4.5
Cardiff[†]	Cable tunnel	Mercia Mudstone (moderately strong mudstone)	McAlpine full face TBM; Big A picks; compressed air	2.44	11
Sutton[†]	Sewer	London Clay	Full face mini-tunnel TBM; picks	1.2	4
M5	Frome Valley Relief Sewer	Mercia Mudstone	Drill and blast	1.83	28 - 47

[†] *After New, 1982*
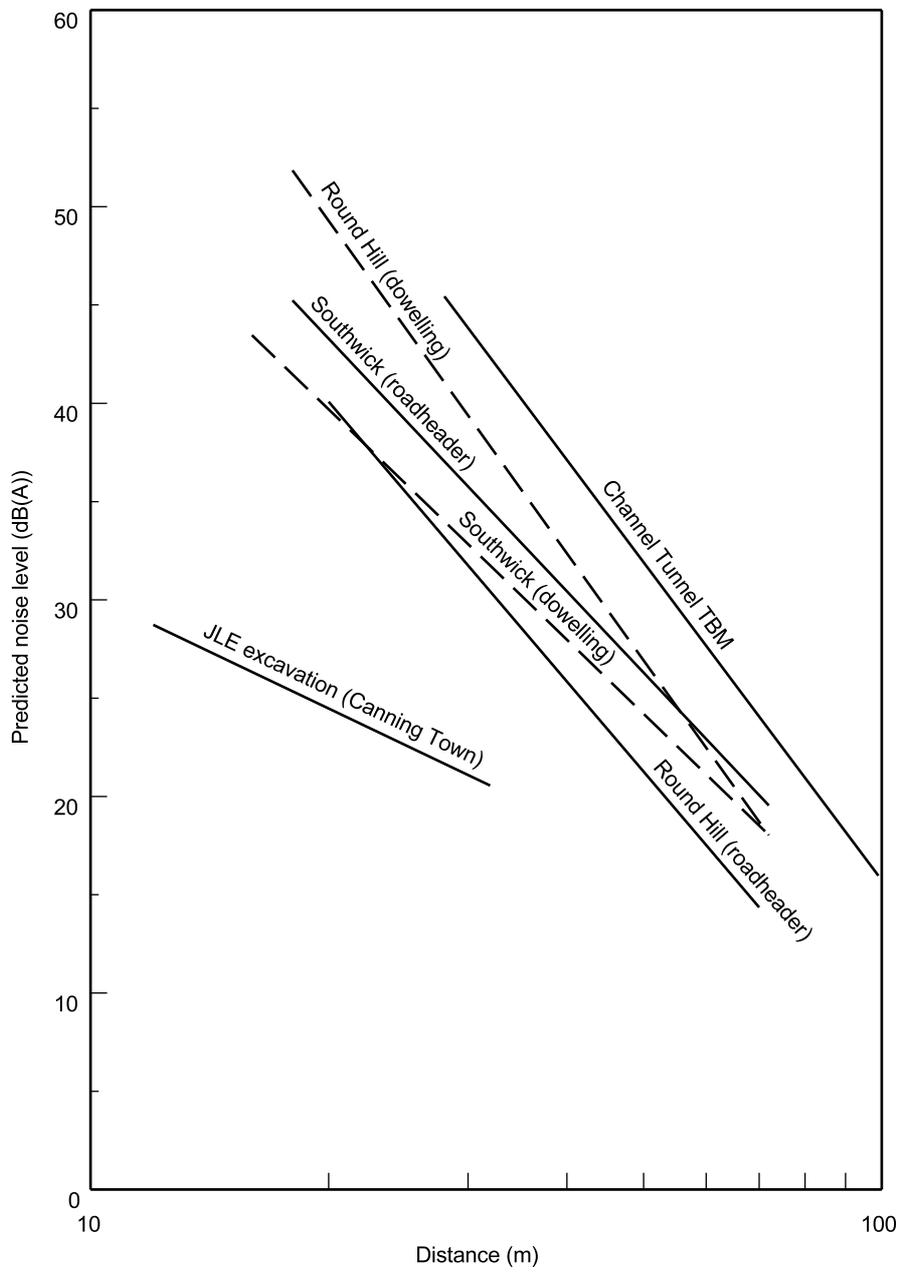
Figure 48a Regression lines fitted to calculated groundborne noise levels for tunnelling operations
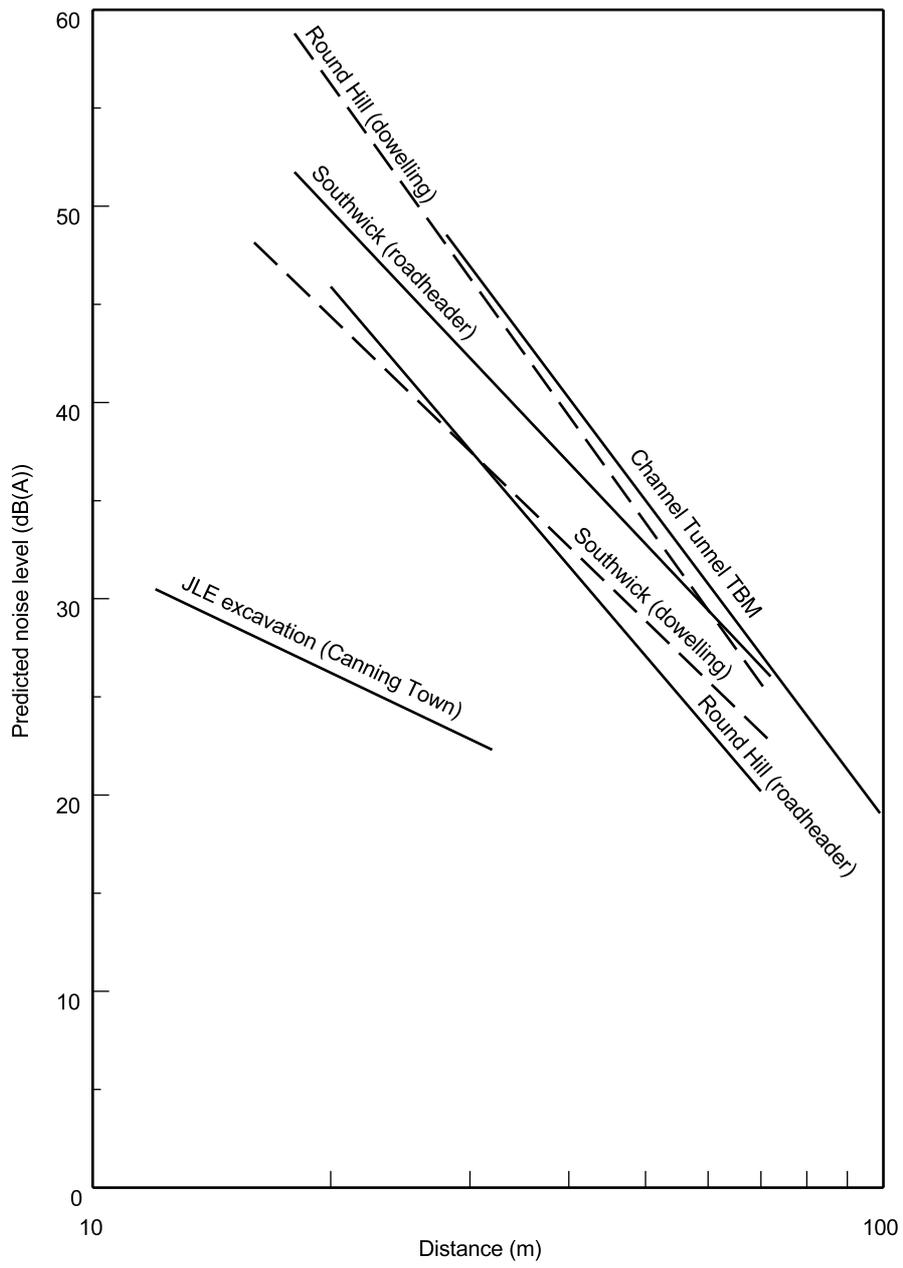
Figure 48b Upper-bound lines fitted to calculated groundborne noise levels for tunnelling operations
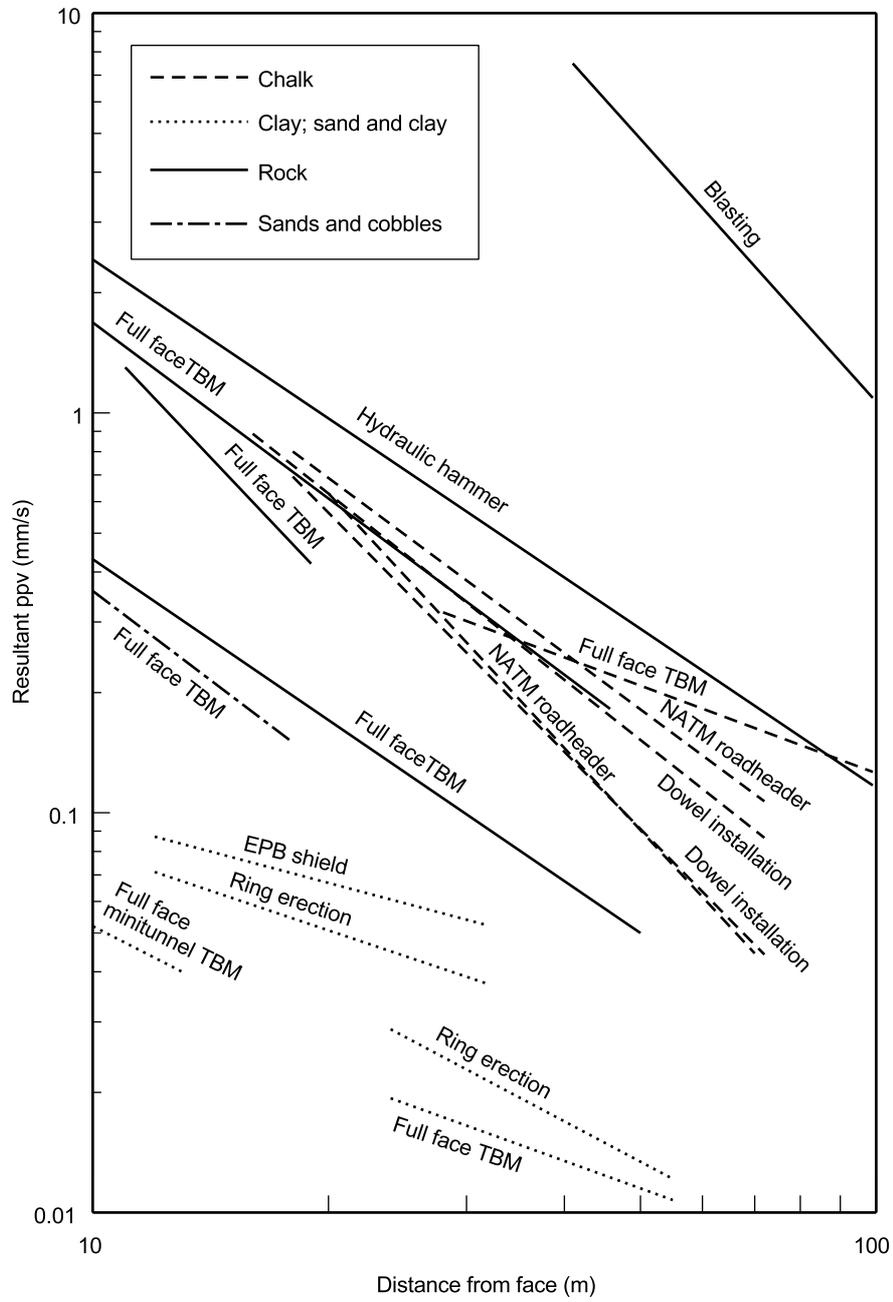
Figure 49 Ground vibration data from tunnelling operations classified according to geology

of vibration than the mechanical tunnel excavation processes. The Frome Valley sewer was excavated though similar geology to the Cardiff cable tunnel so the two may be compared directly. Furthermore, it should be noted that the Frome Valley data may represent a lower bound for blasting vibration since the individual charge weights used for this relatively small tunnel were between 0.6 and 1.2kg. Larger drill and blast tunnels may commonly use greater instantaneous charge weights.

Of the tunnelling data presented here, only blasting produced levels of vibration which might have been sufficiently high to lead to building damage. However, many of the sources produced potentially disturbing levels of vibration up to quite considerable distances from the works.

In general, there appears to be a correlation of the vibration levels caused by mechanised tunnelling with the geological situation (Figure 49). Data from the two New Austrian Tunnelling Method (NATM) sites (Round Hill and Southwick) plot at a similar level to the data from the Channel Tunnel land drive at Holywell Coombe, which used a full face TBM. The common factor in these three sites was that they were all excavated in chalk. Dowel installation at the two NATM sites produced similar levels of vibration to excavation. The data from Cardiff were of a similar magnitude and this tunnel was also excavated in reasonably competent rock.

The Jubilee Line Extension sites were in clay, as was the Sutton sewer tunnel and data from these sites plot somewhat lower than the other data. This difference may reflect the lower amount of energy required to excavate soft ground materials. Activity within the tunnels associated with ring erection generated similar vibration levels to excavation but the lining construction events tended to be of shorter duration. It should be noted, however, that the vibration levels from all these soft ground sites were below the human perception threshold.

The data from Warrington fall approximately between the chalk and clay data. Although excavation was in loose sand, New (1978) reported that the sand contained cobble and boulder sized glacial erratics and it was the impact of the cutters with these that produced the highest vibration levels.

Comparing the data from tunnelling works with other more common sources of vibration helps to put the vibration levels into context and is useful for reassuring the general public who may be concerned about unusual vibration within their properties, particularly when the source of disturbance is not apparent. The chalk tunnelling operations produced vibration levels similar to those recorded from a heavily loaded four axle lorry on a fair road surface and from London Underground trains operating in bored tunnel. The soft ground operations produced levels slightly higher than those recorded from light road traffic.

The method used for the calculation of vibration dose values presented in Figure 47 yields a slightly conservative prediction since the vibration data analysed were the highest levels which occurred and therefore would not exist for the whole 16 hour period. Despite this, and although the levels of vibration were sufficient to be perceptible, comparison of the calculated vibration dose values with guidance in BS 6472 (BSI, 1992a) indicates that these works would not be expected to provoke adverse comment from occupants of residential buildings.

The calculated levels of groundborne noise presented in Figure 48 appear to be the most marginal data when compared with accepted limits. The data from the three chalk sites illustrate clearly that disturbance caused by groundborne noise could be problematic over quite considerable distances from the works. In the worst of the three cases presented here, the 30dB(A) criterion required by APTA would be exceeded at distances of up to some 60m from the source. In the soft ground tunnels the low amplitude vibration levels from the works have limited the calculation of meaningful results to the excavation works at the Canning Town site. Only at distances of less than 12m does the predicted noise level exceed 30dB(A), the lowest target level given by the APTA guidelines (Section 2.2.3). Groundborne noise thus appears to be the factor most likely to be the cause of complaint.

Taking the peak particle velocity data as a whole, an upper bound could be drawn which would provide a useful first estimate for prediction of the vibration levels likely to be generated by future mechanised bored tunnelling works. Taking a line similar to, but enclosing, the upper bound to the data from Godio *et al* (1992) yields the expression:

$$v_{res} = 180 \ r^{-1.3} \tag{24}$$

where v_{res} is the predicted upper bound resultant ppv (mm/s) and r is the slope (shortest) distance (m) from the vibration source to the measurement location. The data further suggest that, in soft ground, this is likely to be excessively conservative and the constant term could reasonably be reduced by an order of magnitude. Similarly, for the prediction of groundborne noise:

$$L_p = 127 - 54 \log_{10} r \tag{25}$$

where L_p is the predicted groundborne noise level in dB(A).

Because Equations (24) and (25) are derived from a limited range of materials it is possible that they may under estimate noise levels caused by tunnelling in stronger rocks, and from very high energy sources such as hydraulic hammers. Therefore care should be taken in their application in these circumstances. Similarly care should be taken in extrapolating these relations over a wider range of distances than that covered by the data from which they have been derived.

8 Conclusions and recommendations

There are two main stages in the assessment of ground vibration problems which may arise on construction sites. These are the prediction of the vibration level at a range of distances from the source and the assessment of the effects of this vibration on structures and their occupants. This report has described a research project primarily focussed on the first stage of this process. The second stage is accommodated by reference to current British Standards, which have been reviewed. Recommendations relating to these two areas are summarised below.

8.1 Prediction of ground vibration

The work reported considerably extends the body of data available for making predictions of ground vibration from vibrating rollers, piling, ground improvement and tunnelling. Based on these data, improved predictors are presented for all but dynamic compaction. The predictor presented here for dynamic compaction is that proposed by Mayne (1985) which has been corroborated by our measurements. The predictors are based on parameters which are readily obtained from manufacturers' published plant specifications, which makes them simple to apply. The predictors are summarised in Table 12.

In common with the findings of past researchers, the effects of material damping on attenuation have not been included in the predictors because in nearly all cases the fit of the model to the data was not significantly improved by its inclusion. The slopes of the attenuation curves are, however, steeper than those expected theoretically from geometric attenuation of body waves in a homogeneous medium (slope of 1/r). Since the majority of the energy from sources of vibration at the ground surface radiates as surface waves (Section 1.3.1), it may be inferred that attenuative mechanisms other than geometric spreading are also effective. For impulsive signals this may be connected with the spreading of the wave packet discussed in Section 1.3.1, which has been observed in the signals acquired from piling operations. Another possible mechanism is that as the energy travels further it will encounter an increasing number of discontinuities, resulting in more mode conversions, reflections and refractions, which will increase the attenuation. Although the attenuation rates varied over the sites examined, the analysis did not identify a correlation between the rate of attenuation and any aspect of the recorded site investigation data. Accordingly the predictors do not include any attenuation function related to site type.

More precise modelling of ground vibration propagation would probably require much more precise knowledge of site stratigraphy and material properties than is normally

Table 12 Proposed empirical predictors for groundborne vibration arising from mechanised construction works

Operation	Prediction equation	Scaling factors (and probability of predicted value being exceeded)	Parameter range
Vibratory compaction (steady state)	$$v_{res} = k_s \sqrt{n_d} \left[\frac{A}{x + L_d} \right]^{1.5}$$	$k_s = 75$ (50%) $k_s = 143$ (33.3%) $k_s = 276$ (5%)	$1 \leq n_d \leq 2$ $0.4 \leq A \leq 1.72$mm $2 \leq x \leq 110$m
Vibratory compaction (start up and run down)	$$v_{res} = k_s \sqrt{n_d} \left[\frac{A^{1.5}}{(x + L_d)^{1.3}} \right]$$	$k_t = 65$ (50%) $k_t = 106$ (33.3%) $k_t = 177$ (5%)	$0.75 \leq L_d \leq 2.2$m
Percussive piling	$$v_{res} \leq k_p \left[\frac{\sqrt{W}}{r^{1.3}} \right]$$	For piles at refusal: $k_p = 5$ For piles not at refusal: $1 \leq k_p \leq 3$, depending on soil type (Table 9)	$1 \leq L \leq 27$m $1 \leq x \leq 111$m (where $r^2 = L^2 + x^2$) $1.5 \leq W \leq 85$kJ
Vibratory piling	$$v_{res} = \frac{k_v}{x^\delta}$$	$k_v = 60$ (50%) $k_v = 126$ (33.3%) $k_v = 266$ (5%)	$1 \leq x \leq 100$m $1.2 \leq W_c \leq 10.7$kJ $\delta = 1.3$ (all operations) $\delta = 1.2$ (start up and run down) $\delta = 1.4$ (steady state operation)
Dynamic compaction	$$v_{res} \leq 0.037 \left[\frac{\sqrt{W_h}}{x} \right]^{1.7}$$		$5 \leq x \leq 100$m $1.0 \leq W_h \leq 12$MJ
Vibrated stone columns	$$v_{res} = \frac{k_c}{x^{1.4}}$$	$k_c = 33$ (50%) $k_c = 44$ (33.3%) $k_c = 95$ (5%)	$8 \leq x \leq 100$m
Tunnelling (groundborne vibration)	$$v_{res} \leq \frac{180}{r^{1.3}}$$		$10 \leq r \leq 100$m
Tunnelling (groundborne noise)	$$L_p = 127 - 54 \log_{10} r$$		$10 \leq r \leq 100$m

A is the maximum amplitude of drum vibration (mm); L is the pile toe depth (m); L_d is the vibrating roller drum width (m); L_p is the room octave band sound pressure level (dB) ; n_d is the number of vibrating drums; r is the slope distance from the pile toe (m); v_{res} is the resultant peak particle velocity (mm/s); W is the nominal hammer energy (J); W_c is the energy per cycle (kJ); W_h is the potential energy of a raised tamper (J); x is the distance measured along the ground surface (m);

available. Dynamic numerical analysis calibrated by field vibration data might provide a further insight into this problem. This is a possible area for future research.

The predictors are valid up to a distance of about 100m from the vibration source. This encompasses the distances at which ground vibration is likely to be perceptible at most sites, although the effects of large vibratory rollers and piling may be perceptible at greater distances. Extrapolation much beyond this distance is not recommended although it will generally provide a conservative estimate.

Probabilistic predictors are presented for vibratory compaction and vibratory piling. For the other activities predictors presented as upper bounds to the data and therefore are expected to be generally conservative. However it should be remembered that for vibrating rollers there are likely to be transients at starting and stopping which may generate particle velocities which can be twice as large as for steady state operation. Significantly lower speeds than the 1.5 to 2.5km/h specified will also result in higher particle velocities. The implications of this are that rollers should not be started, stopped, or the direction of travel reversed near sensitive structures. In the case of vibratory piling, the predictor includes the effects of resonance, which did not always generate the highest vibration levels, particularly at the shorter distances. The 'resonance free' variety of pile vibrator should, however, be used whenever possible if there are sensitive structures nearby.

In addition to those construction activities for which vibration predictors have been proposed, data from a number of other activities have been acquired during the current study. These are presented in Figure 50 and may be of use for obtaining an indication of possible vibration levels from similar activities on other sites.

8.2 Assessment of effects of ground vibration.

Standards have been reviewed (Section 2) which cover the three main criteria against which assessment of ground

vibration prediction may be required. These are structural damage, perceptible intrusion and groundborne noise (audible intrusion). The relevant documents for use in the UK are:

For damage assessment
BS7385: Part 2, *Evaluation and measurement for vibration in buildings. Guide to damage levels from groundborne vibration* (BSI, 1993) and BS 5228 : Part 4 : 1992, *Code of practice for noise and vibration control applicable to piling operations* (BSI, 1992). Both are applicable to all the sources considered in this report and are discussed in Section 2.1.1. The later document, BS7385, which is the less conservative and also gives levels for the three damage thresholds, *cosmetic, minor* and *major*, appears to be the most useful. However BS5228 does include a table showing threshold values for a wider range of structures. In both standards the levels for intermittent vibration are of individual components of particle velocity rather than resultants. It is recommended that the levels are halved for continuous vibration.

For the assessment of perceptible vibration intrusion
BS 6472 : 1992, *Evaluation of human exposure to vibration in buildings (1Hz to 80Hz)* (BSI, 1992a) gives guidance on threshold values for human perception and the higher levels above which complaints become more likely in a variety of environments. It also gives guidance on the calculation of vibration dose values, which depend on both the magnitude and the duration of the vibration. As has been discussed in Section 2 there is currently some debate over the application of vibration dose values to construction operations.

For the assessment of audible intrusion (groundborne noise)
There is no UK guidance on this so recourse is often made to the APTA guidelines described in Section 7.
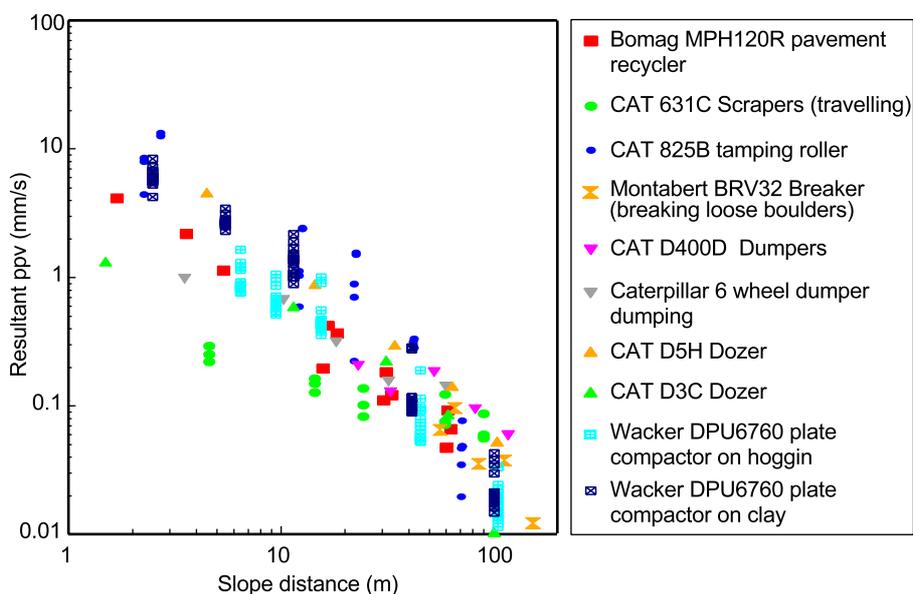

Figure 50 Data acquired from miscellaneous operations on highway construction sites

8.3 Recommendations for future work

The possible use of numerical modelling to help investigate the mechanisms of vibration attenuation in the ground is mentioned above.

8.3.1 Low-frequency ground vibration

The current research has highlighted the need for a more comprehensive investigation of low-frequency (below 4.5Hz) ground vibrations due to plant movement on soft ground. Such vibrations have been experienced on the A13 construction crossing Rainham Marshes and similar vibrations have caused complaint on the Second Severn Crossing approach roads. Ground vibration data could be acquired from suitable construction sites or pilot scale tests. The magnitude of the effect from a variety of operations would be assessed to determine the zone within which complaint might be experienced. The database acquired during the current programme would also be extended to enhance the reliability of prediction.

8.3.2 Mitigation of ground vibration

Buried cut-off barriers offer a promising method of isolating sensitive structures from the potentially damaging or intrusive effects of groundborne vibration. The production of guidance on the design and use of such barriers could be addressed by reviewing published material and by performing full-scale trials at suitable field sites. In such trials, the effects of different materials, geometries and vibration sources would be assessed. The use of lightweight fill embankments as vibration barriers could also be investigated. Comparative site measurements could be made on existing structures built of polystyrene and PFA to minimise ground loading.

9 Acknowledgements

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Appendix A: Controlled study of vibrating rollers

This appendix describes the practical aspects of the research undertaken during the controlled trial which investigated vibration levels arising from compaction plant. The design and construction of the earthwork are discussed, followed by the details of the tests undertaken with each item of plant. The analysis and interpretation of the data are described in the main body of the report.

A.1 Selection of the trial site

A location was required for the trial where the ground surface was reasonably flat and level; where the general ground conditions were known without undertaking a ground investigation; where it was possible to measure vibration over a distance of 100m without topographical changes; where access was possible for plant and construction materials; and where there were minimal levels of ambient environmental vibration.

A site was identified within the grounds of the Transport Research Laboratory (TRL) at Crowthorne. It was on the edge of a football field, so the dimensions and topography were ideal (Figure A1). The ground conditions were known from the site investigation work undertaken at the time of construction of TRL's test track (Lewis, 1954) and the upper soil horizons were known to be fill material, placed to achieve a level playing surface. To the east of the test site, the TRL test track is located in a shallow cutting. On the west side of the trial site runs the B3430 road. It was desirable that the most distant location at which vibrations were to be measured was not close to the road, to prevent the vibrations caused by the plant operating being obscured by vibration from road traffic. It was also required that any potential for reflection or diffraction from the cutting slopes was minimised. Consequently, the test structure was oriented so that the most distant geophone location fell approximately midway between the test track and the B3430.

A.2 Design of the trial earthwork

The earthwork was required to be of sufficient length and width that it was able to accommodate plant representative of the largest categories currently in use in the UK. It also had to be constructed with a sufficient thickness of fill that the compaction effect of the largest plant would be contained within the fill material (Parsons, 1992). Two separate earthworks were to be constructed, each using a different fill, so that the effect of fill material on the level of vibration could be investigated.

An embankment was planned to be constructed with a total height of 1.5m, founded at 0.5m below the existing ground level, in accordance with the common practice of removal of the topsoil ahead of construction of
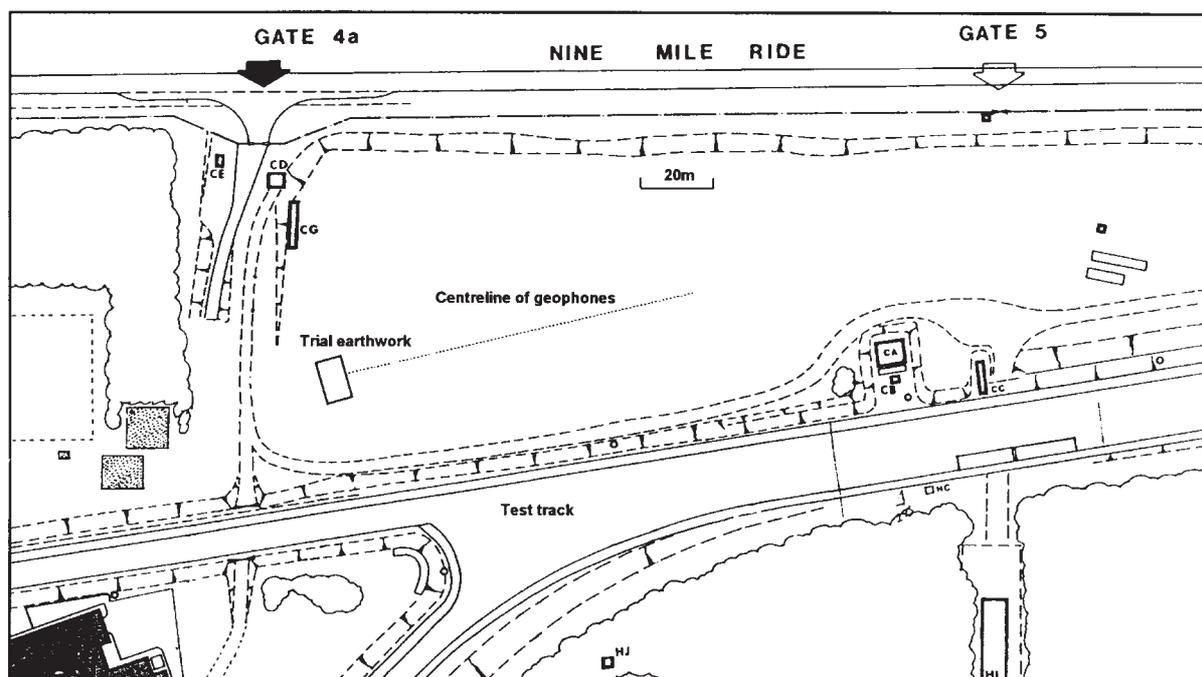

Figure A1 Location plan of trial site at TRL

embankments. This approach would have the advantage of reducing the height of the embankment, thereby reducing the total volume of fill required and improving safety for the plant operatives. The structure would have a length of 10m, excluding the access ramps required at both ends. Two 4m wide embankments, each of a different material, would be constructed side-by-side, so that the amount of imported fill required was reduced, by eliminating two of the side slopes.

Two commonly used fill materials with different material properties were required, both of which could be compacted to the requirements of the Specification for Highway Works (SHW) (MCHW1, 1993) by the largest possible range of plant. The materials which were selected were a Class 1A (well graded granular) general fill and a Class 2A (wet cohesive) general fill. The former was as-dug hoggin sourced from Eversley, Hampshire and the cohesive material was London Clay from the Isle of Sheppey. Smooth drum rollers would not normally be allowed for compaction of wet cohesive materials and tamping foot or grid rollers should be used. This is because shear surfaces may develop in the clay, possibly leading to premature failure (Whyte and Vakalis, 1988). This was not considered to be a problem for this experiment since the embankment was relatively small and was only required to remain serviceable for a few months.

A.3 Construction of the trial earthwork

At an early stage of the construction it was decided to increase the foundation depth from 0.5m to 1m, although the same total thickness of fill was to be maintained. This would significantly reduce the volume of fill required and simplify the construction process by reducing the side slopes and ramps. Excavation revealed the soil profile illustrated in Figure A2. Below the forest floor deposits, the soil in the root zone was very soft mottled clay. This layer was at the depth intended to be the foundation depth for the embankment, but it was too soft to traffic, having an *in situ* California Bearing Ratio (CBR) of less than one per cent. During construction, the excavation was therefore continued until a suitable foundation material was encountered. This occurred at a depth of 1.5m, in a grey fine sand which had *in situ* CBRs of typically six per cent.

Before placing the fill, holes approximately 200mm x 200mm x 150mm deep were excavated, into which uniaxial vertical geophones were installed beneath the centre of each side of the structure. These were positioned such that the top of each was flush with the foundation level. The geophones were mounted on 80mm long spikes pushed into the undisturbed ground and the excavated material was then placed by hand and tamped around them. The cables from the geophones, fitted with an armoured duct to protect them during construction, were brought out to one side of the excavation.

The need to remove the soft ground resulted in a deeper excavation than had been designed (Figure A3a). Therefore, to ensure that the volume of imported fill was sufficient to complete the construction, an additional supply of Class 1A granular fill was used for the first three layers across the whole base of the excavation (Figure A3b). Each layer of
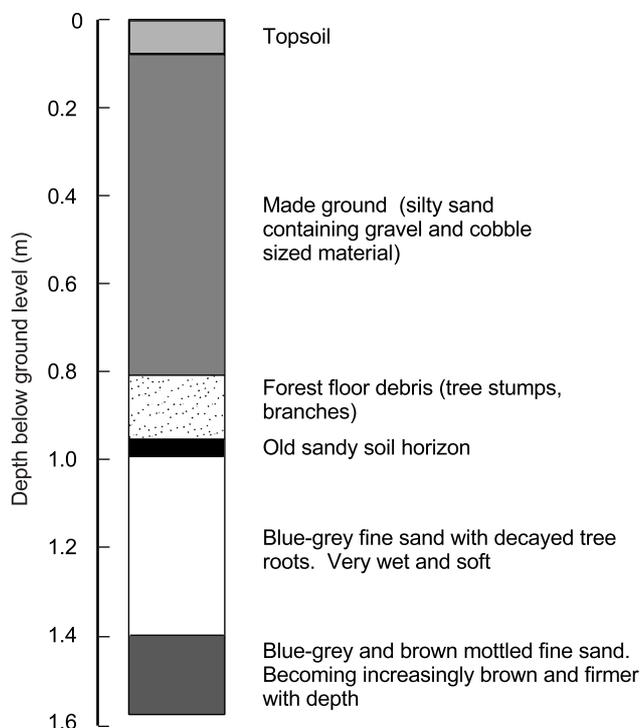

Figure A2 Soil profile at TRL trial site

the fill was placed according to the requirements of the SHW. A Bomag BW100AD smooth drum tandem roller was used and two 100mm thick layers were placed, each being compacted with five passes of the roller. In a number of locations the natural wet sand from beneath boiled through the compacted material. The third and final layer of this material was therefore compacted with only two passes with the vibrator operating and two further passes using the roller as a dead weight roller, which helped to seal the base of the excavation.

Above the three layers of granular fill placed at the base of the structure, subsequent layers were placed with one side of the construction of hoggin and the other side of clay (Figure A3c). The clay was placed on the wetter side of the excavation, since this would have the lower permeability and therefore would limit any wetting up of the fill from beneath. Before placing the clay, the stockpile was rotavated (Plate A1) to break up large blocks into a size appropriate for the compaction plant and to ensure a uniform moisture content existed. Soil testing required by the SHW was undertaken to ensure that the imported fill materials met the acceptability for earthworks requirements for the particular class of fill. The results are summarised in Tables A1 and A2 and Figure A4. The dry and wet categories for cohesive fills are defined by the moisture content relative to the plastic limit. The moisture content of the upper layers of fill fitted into the dry category, even though the material as delivered would be classified as wet.

Placing and compaction of the fill required 13 lifts, which brought the fill up to the original ground level. Construction from 630mm below ground level was completed using a Bomag BW120AD-3 because of a
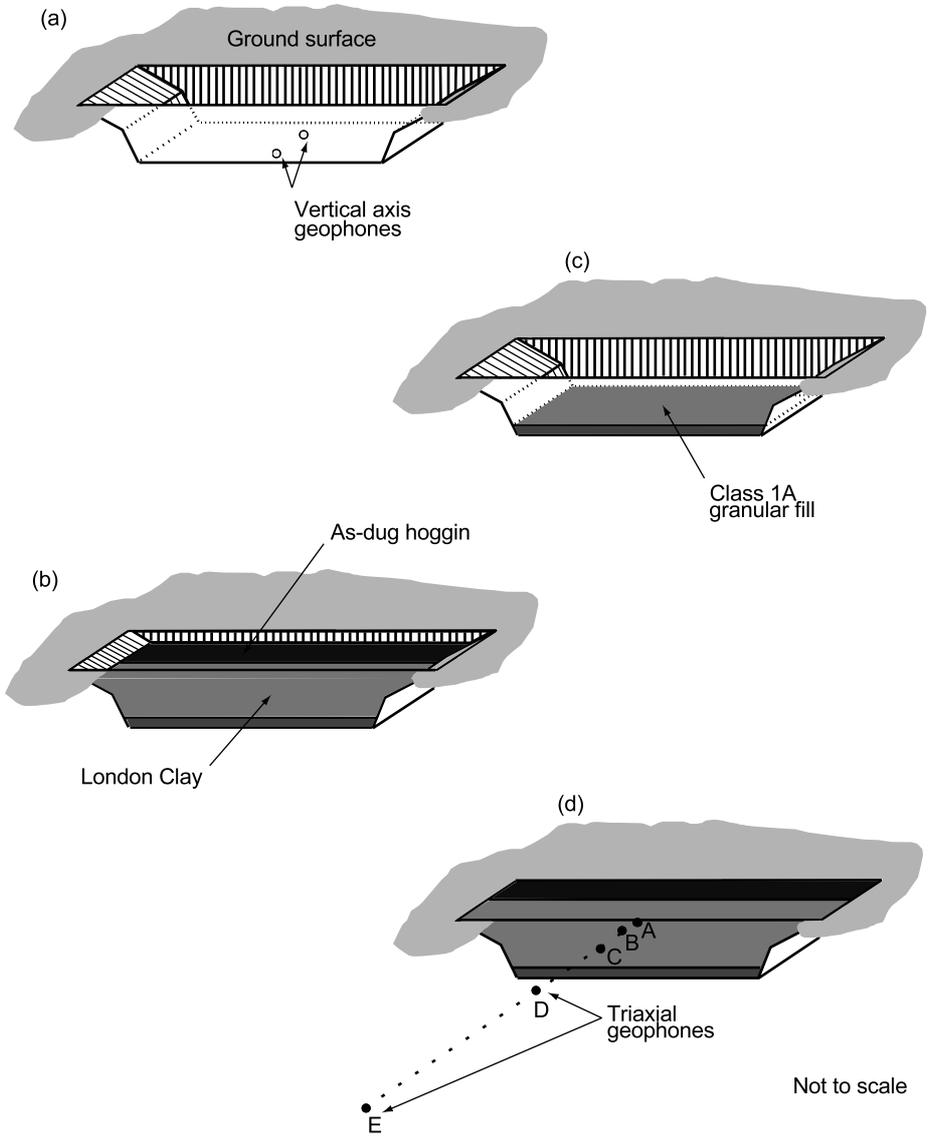
Figure A3 Construction sequence of TRL trial earthwork

Plate A1 Rotavation of clay fill

Table A1 Classification test data determined for London Clay (as delivered to the test site)

Fill type	Moisture condition value	Moisture content (%)	Plastic limit (%)	Liquid limit (%)	Plasticity index (%)	Optimum moisture content (%)
Brown London Clay	13.6 13.3	31 34	29	78	49	27 (determined by 2.5kg rammer method)
Grey/brown London Clay	16.8 15.8	29 31	28	78	50	

Table A2 Classification test data for hoggin (as delivered to the test site)

Fill type	Moisture condition value	Moisture content (%)	Uniformity coefficient	Optimum moisture content (%)
As-dug hoggin	12.4 10.9 13.2	9 10 10	160	7.5 (determined by vibrating hammer method)
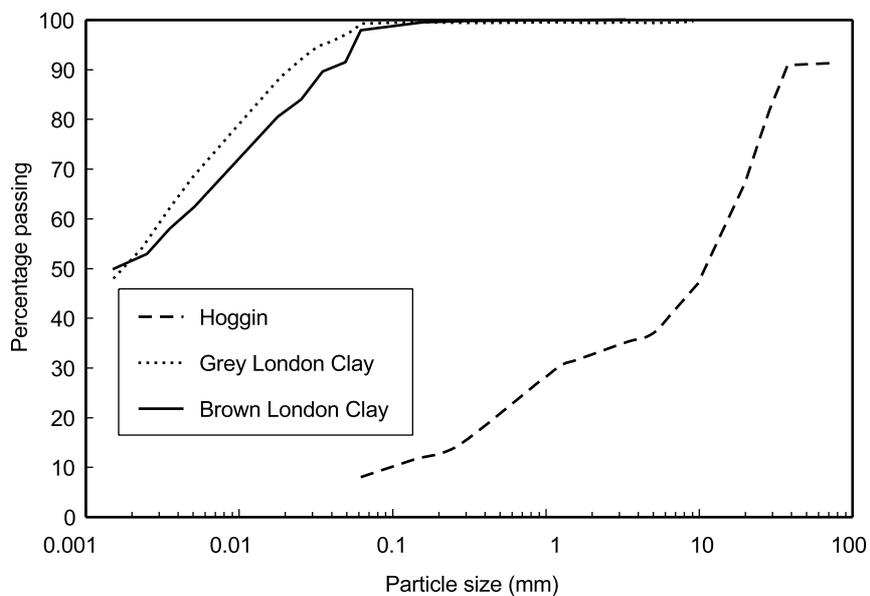
Clay	Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	Cobbles
	Silt			Sand			Gravel			

Figure A4 Particle size distributions for the fills used in the trial

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failure of the Bomag BW100AD roller after compaction of the tenth clay layer.

Construction ceased when the compacted fill level had reached original ground level. The earthwork was not extended to be partially above ground, as had been originally intended, for the following reasons. During construction it became evident that the presence of a ramp would present an access problem for smooth drum rollers on the clay because of a lack of traction. Moreover, a constant speed of travel along the whole length of the fill would be more easily achieved if there was no ramp to negotiate. Additionally, operation of the plant at ground level could be a worse case in terms of levels of vibration than if the structure was continued to above ground level. Plate A2 illustrates the form of the earthwork as it neared completion.

During construction, vibration measurements were made from the buried geophones for the final compaction pass of each layer. The depth below ground level of each layer was recorded after compaction.

Three standpipe piezometers were also installed so that the ground water level could be monitored. One of the differences between this trial and the measurements made on construction sites was that, for the controlled experiment, the propagation of vibration would be through the same medium for each item of plant, except that changes could occur due to differences in the ground water level. Three piezometers were installed at various depths so that this could be monitored throughout the trial.

The first borehole was driven to place the piezometer tip at 10.15m below ground level. During this drive, a very wet layer of silty sand was encountered at a depth of approximately 6m. It was therefore intended to install the second piezometer at a depth to monitor this apparently perched water table. The second piezometer was installed approximately 5m away from the first, but the wet layer was not present. Water was encountered at 8m and the piezometer was installed at 8.6m. The third piezometer

was installed to monitor the water level at a depth encountered during excavation of the trial site, and was located at a depth of 1.47m. The ceramic tips of the piezometers were each installed in a cell of single sized sand. The annuli between the access tubes and the borehole walls were backfilled with bentonite pellets and the access tubes filled with water. Water levels were recorded each day that vibration measurements were undertaken. The observed changes in the water levels were sufficiently small that the conditions were considered to have been constant throughout the trial (Figure A5).
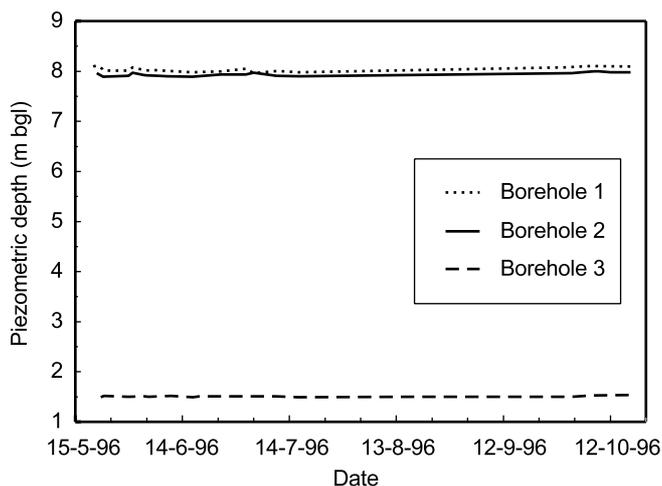
Figure A5 Piezometer data for the duration of the TRL field trial

A.4 Selection of the plant for trials

The selection of the rollers used for the trials was based on the range of plant specified as suitable by Table 6/4 of the SHW (MCHW1, 1993) for compacting the two fills chosen. This document categorises vibratory rollers on the mass per unit width of the vibrating drum, which has been

Plate A2 Construction of test earthwork near completion

shown to be a good practical measure of the performance of vibrating rollers (Parsons, 1992). A method specification for compaction is provided by SHW which specifies the compacted layer thickness and the number of passes required, based on the requirement of achieving a minimum 10 per cent air voids. The relevant sections of Table 6/4 are reproduced in Table A3.

Initially six items of plant were tested, selected so that the maximum possible range of the categories given in Table A3 was represented, within availability and financial constraints. Further plant became available at later dates, which allowed some additional testing to be undertaken, but the amount of data from these later plant were restricted for various reasons, as discussed in Section A6.

All plant tested were double drum rollers except for those with reference numbers of 7 and 9 assigned according to the SHW, which were single drum self propelled rollers. Vibrating plate compactors are also included in SHW; one of these was available and was tested following the same procedure as for the rollers. Details of all the plant from which groundborne vibration was recorded during both the construction and testing phase are summarised in Table A4 and illustrated in Plates A3 to A13.

A.5 Testing methodology

The primary objective of this phase of the research was to acquire data from which it would be possible to predict levels of vibration which may arise from compaction works, so that assessments of the potential for damage to property and intrusion can be made. To satisfy this requirement would require that the maximum level of vibration which could arise from any particular piece of plant was determined. This might be affected by both the method of compaction and the characteristics of the fill. Consequently, vibration was recorded from a succession of compacting passes with each item of plant and the changes which the fill underwent were measured by use of a nuclear density gauge. A variety of manoeuvres with the

plant, in addition to the normal pass-bys were also recorded to assess the effects of these on the predicted vibration level. This section describes the aspects of the testing programme which were common to all plant tested. Section A6 describes variations to this procedure which occurred for the individual items of plant.

The data acquisition unit enabled signals from up to sixteen transducers to be recorded simultaneously. Triaxial arrays of geophones were positioned at distances of 1m, 4m, 10m, 40m and 100m from the edge of the test structure. For all the main phase of testing the geophones were located in the same positions, which were all to one side of the test structure. Consequently, the distances of each of the geophones from rollers operating on the hoggin were always greater than when the rollers were on the clay. Vibration propagating from sources on the hoggin encountered an additional acoustic impedance mismatch at the hoggin/clay interface, which was not encountered by vibration from rollers operating on the clay (Figure A6). However, using the same geophone locations for measuring vibration during the compaction of both fills eliminated any potential effects on the recorded vibration caused by local variations in ground conditions.

Following completion of the construction of the test area, for each item of plant tested, the top layer of fill was rotavated to a depth approximately equal to the layer thickness specified by the SHW for the particular category of plant being tested. Rotavation broke up the compacted fill, into a state similar to that in which it would normally be placed, ready for compaction trials to commence. For each item of plant tested on each of the two fills, the following testing procedure was undertaken.

A strip of fill, centered on the centre line of that material, was compacted with one pass of the roller, operated at its normal operating speed and in its normal mode and the resulting ground vibration was recorded. A *Troxler* nuclear density gauge (NDG), calibrated by sand replacement tests on the two fill materials, was then used to determine the bulk and dry densities and the moisture content of the fill. The measurement depth was the same as that corresponding

Table A3 Extract from Table 6/4 from the SHW (MCHW1,1993) defining compaction requirements for the fill used

Ref No.	Category (mass, M, per metre width of vibrating roll; kg)	Method 1*		Method 2**	
		Layer thickness (mm)	Number of passes	Layer thickness (mm)	Number of passes
1	$270 < M \leq 450$	Unsuitable		75	16
2	$450 < M \leq 700$	Unsuitable		75	12
3†	$700 < M \leq 1300$	100	12	125	12
4†	$1300 < M \leq 1800$	125	8	150	8
5	$1800 < M \leq 2300$	150	4	150	4
6†	$2300 < M \leq 2900$	175	4	175	4
7†	$2900 < M \leq 3600$	200	4	200	4
8	$3600 < M \leq 4300$	225	4	225	4
9†	$4300 < M \leq 5000$	250	4	250	4
10	$5000 < M$	275	4	275	4

* Method 1 is suitable for wet cohesive (Class 2A) fill.
** Method 2 is for well graded granular (Class 1A) and dry cohesive (Class 2B) fill. The required number of passes may be halved for tandem vibrating rollers.
† Indicates categories tested.

Table A4 Summary of plant used during the controlled trial

Plant model	Plant type *	Drum width (m)	Mass per metre width (kg/m)		High setting			Low setting		
			Front	Rear	Ampli-tude (mm)	Freq-uency (Hz)	Centri-fugal force (kN)	Ampli-tude (mm)	Freq-uency (Hz)	Centri-fugal force (kN)
Ingersoll-Rand SD-150D	Single drum roller	2.14	4367	-	1.77	26.5	245	0.89	26.5	123
Dynapac CA301	Single drum roller	2.13	3150	-	1.72	30	249	0.84	33	146
Hamm 2422DS	Single drum roller †	2.14	2920	-	1.76	30	350	0.7	42	300
Bomag BW 161 AD	Tandem roller	1.68	2680	2740	0.91	30	58	0.43	45	62
Ingersoll-Rand DD-65	Tandem roller	1.40	2300	2400	0.59	55	80	0.29	55	40
Bomag BW 135 AD	Tandem roller	1.30	1330	1390	0.4	60	37	0.4	50	26
Bomag BW 120 AD-3	Tandem roller	1.20	1130	1130	0.53	66	41	0.53	55	29
Bomag BW 100 AD-3	Tandem roller	1.00	1200	1200	0.52	66	34	0.52	55	23
Rammax RW2400	Tandem roller †	1.20	1000	1000	n/a	41	84			
Benford TV75	Tandem roller	0.75	920	920	0.5	50	9.8			

					Front drum			Rear drum		
Bomag BW 161 AD-CV *Variomatic*	Tandem roller	1.68	2400	2380	0.92	35	39	0.31	47	79

Wacker DPU 6760	Vibrating plate	1075 x 800mm	711kg/m² **		n/a	56	66.5			

*   All rollers used for this research were self-propelled models; all were smooth drum models except † which were tamping rollers
** Mass per m² of base plate
n/a = not available
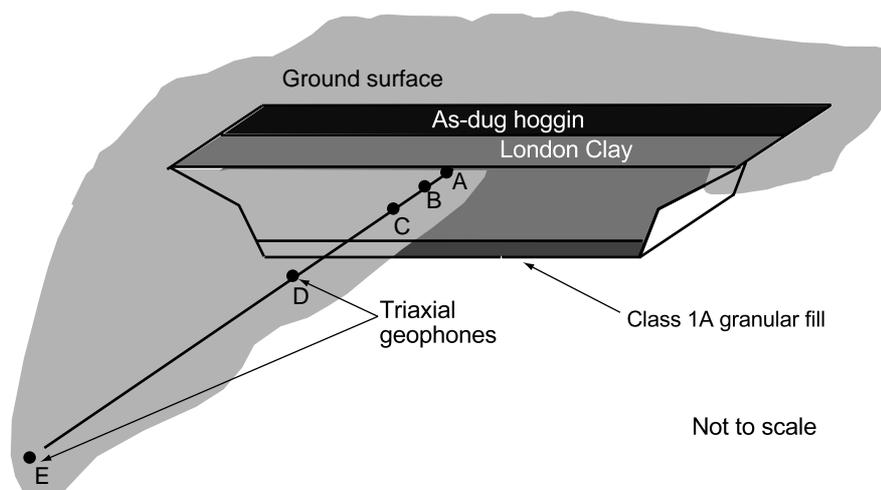
Figure A6 Schematic arrangement of test structure

Plate A3 Ingersoll-Rand SD-150D (4367kg/m width)

Plage A4 Dynapac CA 310 (3150kg/m width)

Plate A5 Hamm 2422 DS tamping roller (2920kgm width)

Plate A6 Bomag BW 161 AD (2680/2740kg/m width)

Plate A7 Ingersoll-Rand DD-65 (2300/2400kgm width

Plate A8 Bomag BW 135 AD (1330/1390kg/m width)

Plate A9 Bomag BW 120 AD-3 (1130/1130kg/m width)

Plate A10 Bomag B W 100 AD3 (1200/1200kg/m width)

Plate A11 Rammax RW2400 (1000/1000kg/m width)

Plate A12 Benford TV75 (920/920kg/m width)

Plate A13 Bomag BW 161 AD-CV Variomatic (2400/2380kg/m width)

Plate A14 Accelerometers mounted on roller drum

to the compacted layer thickness specified by SHW. Measurements were made with the NDG at three locations on the centre line of the trafficked strip.

Following the NDG measurements, the same strip of fill was compacted with a second pass of the roller, maintaining the same line as the first pass, while the ground vibration was recorded again. For this pass, the roller was reversed over the fill from the opposite end of the test bay to which the first pass had been made. A further set of measurements of the fill properties was made with the NDG. This procedure was repeated with NDG measurements made after each of the first 5 passes and following the seventh, tenth and fifteenth passes. To comply with Table 6/4 of SHW would not require 15 passes with any of the rollers tested, but compaction was continued to this degree so that any resulting changes in the levels of vibration could be investigated.

For each pass, the peak particle velocity (ppv) recorded by the buried vertical geophone was transferred to a spreadsheet. This was used to monitor progress of the compaction by plotting the measured ppv against the number of passes, which showed a general increase in ppv with pass number. Compaction was continued until there was no further increase in ppv with further passes. For some combinations of roller and fill the direction of travel appeared to affect the vibration level, with the ppv being greater for passes when the roller was travelling in one direction than when travelling the other way.

Following the measurements made of vibration arising from normal compaction passes, vibrations caused by other movements which might occur on construction sites were measured. These included the transient vibrations arising during starting up and shutting down the vibratory mechanism while the roller was travelling; the effect of changing the travelling direction from forwards to reverse, without shutting down the vibratory mechanism; and operating only one drum of tandem rollers. Additionally, the vibration arising from operating at different travel speeds and different vibration amplitudes and frequencies were investigated, where the plant permitted.

Direct measurements of the vibration of the drum were made for some of the rollers. These required the use of accelerometers, which were not available at the start of the tests, so only a limited number of data were acquired. The geophones were not suitable for these measurements because their sensitivity limited the full scale deflection of

the digital system to ±200mm/s at the minimum gain of unity. The accelerometers had a lower sensitivity, enabling up to approximately ±500mm/s to be recorded. Accelerometers were attached to the vibrating drum of the rollers by a heavy steel bracket, which was glued to the drum using a thin layer of *Plastic Padding*. The triaxial array was then bolted on to this (Plate A14). Accelerometer arrays were also used to measure vibration of the fill, as close as was practicable to the drum, so that the transfer of energy from the drum to the ground could be considered. However, the static case is somewhat different from the case when the roller is travelling. When the roller is stationary, the oscillation cyclically loads the same soil mass, whereas when the roller is travelling, the roller continuously encounters different soil elements (Yoo and Selig, 1979).

A.6 Testing specific to each item of plant

The preceding discussion has described the general experimental procedure used to investigate all the items of plant tested. The following Sections describe variations to the general procedure which were used for each piece of plant. This information is presented for completeness: although the experiment was nominally controlled, there were sufficient anomalies, which may affect the interpretation of the results, that it is necessary to present these details in full. The sections are presented in chronological order of the testing (Table A5).

A.6.1 Benford TV75

The first and smallest roller studied was a Benford TV75. Small pieces of plant are relatively cheap to hire so this was used to verify the experimental method before hiring larger plant. Successive passes of the roller were made with the roller travelling in opposite directions along the fill. Alternate passes were made travelling in opposite directions from alternate ends of the test area. The data from the buried geophones showed the vibration level to be approximately constant after eight passes on the hoggin. On the clay, the trend was still an increase in ppv with each pass, even after 15 passes, although the data were more erratic than on the hoggin. The surface of the fill became very uneven, which might account for some of the variation.

Table A5 Chronology of plant testing

Plant	Date tested	Controllable variables investigated
Benford TV75	23/5/96 - 29/5/96	Single drum operation
Dynapac CA301	4/6/96 - 5/6/96	High and low amplitude; travel speed
Bomag BW135AD	17/6/96 - 19/6/96	High and low amplitude; travel speed; single drum operation
Ingersoll-Rand DD65	25/6/96 - 1/7/96	High and low amplitude; travel speed ;single drum operation
Ingersoll-Rand SD150	2/7/96 - 4/7/96	High and low amplitude; vibration frequency
Bomag BW161AD-CV *Variomatic*	9/7/96 - 11/7/96	Single drum operation; travel speed; angle of vibration orientation
Wacker DPU6760	16/7/96 - 17/7/96	None
Hamm 2422DS	4/10/96	None
Rammax 2400	4/10/96	None
Bomag BW120AD-3	15/9/97	[Effect of fill type - separate experiment]
Bomag BW161AD	23/9/97 - 24/9/97	Travel speed

A.6.2 Dynapac CA301

The second roller studied was a heavy single drum Dynapac CA301. During construction of the test structure, the fill had been compacted to the requirements of SHW, but it was considered that continued trafficking, throughout the duration of the experiment, might cause some additional compaction of the material below the rotavated layer, particularly when using the larger plant. This would mean that the fill conditions were different for each successive piece of plant, if progressively larger plant were used. To ensure that the conditions were as constant as possible for all subsequent pieces of plant on trial, the test area was compacted with eight passes of the Dynapac CA301 before rotavating the top layer of fill for testing. Data presented by Parsons (1992) demonstrated that the change in the amount of compaction achieved by each pass would be expected to have reduced significantly after approximately eight passes. The number of passes was kept to a minimum on the hoggin because over compaction caused the larger particles to fragment.

Tests with the Dynapac CA301 were first carried out on the hoggin. The same approach was taken to testing as used for the Benford TV75, making alternate forward and reverse passes over the fill. The number of normal passes of the hoggin was limited to ten to avoid over compaction. While the vertical ppv at foundation level remained reasonably constant for successive passes made with the roller travelling in the same direction, there was a noticeable difference between the roller travelling forward and in reverse, with the vibration being consistently higher for the reverse passes. The roller was turned around and forward and reverse passes were made and the same trend was demonstrated.

The effect of travel direction on ppv was not exhibited while trafficking the clay. The data showed a smooth increase in ppv with pass number, leveling off after 11 passes. The ppv recorded during the tenth pass was anomalously low. For this pass the driver did not manage to stay exactly on course, so the roller was partially on uncompacted fill.

A.6.3 Bomag BW135AD

Two sets of measurements were made on the hoggin using the Bomag BW135AD. For the first set, ten passes were made, travelling in alternate forward and reverse directions. It was apparent that the fill was too wet to compact, since fines boiled through the coarser material at a number of locations. The vertical ppv at foundation level was reasonably constant for this part of the experiment, although for the first five passes, the ppv when travelling in reverse was higher than when travelling forwards. For the final five passes, there was no difference.

The hoggin was rotavated and left to dry out for a day, before being rotavated a further time and then tested again. The moisture content reduced by approximately three per cent during this period. Vibration levels recorded were significantly higher than had been recorded while the hoggin was wet. There was no difference in vibration level correlated with the travel direction.

For the first six passes on the clay, there was a steady increase in the ppv with pass number, and no difference between travelling forwards or in reverse. For subsequent passes, the vibration arising while travelling forwards was considerably in excess of the levels which occurred while in reverse. The surface of the clay had developed an uneven profile during the trafficking and one possible explanation of the difference between forwards and reverse passes might have related to effects caused by passing over the uneven profile in different directions. To investigate this, the profile was smoothed and compacted again, before a further forward and reverse pass was performed; however the same effect recurred.

This was the first roller for which the accelerometers were available to allow direct measurement of the vibration of the drum.

A.6.4 Ingersoll-Rand DD65

The first measurements made with the Ingersoll-Rand DD65 were undertaken on the clay in two stages, yielding two sets of data. All tests were performed with the engine speed set to the maximum number of revolutions per minute; the travel speed was governed by a separate control. Towards the end of the first set of tests, it became apparent that the vibratory mechanism in the rear drum had not been functioning when it was set to high amplitude, ie the setting used for the majority of the tests. This was subsequently rectified and a further set of tests was undertaken. Consequently data are available from the DD65 operated on clay both as a tandem roller and also with only the front drum vibrating.

With only the front drum vibrating, the vibration level recorded did not show any dependence on direction of travel for the first five passes. Subsequently the vibration was slightly greater when travelling in reverse than when travelling forwards. When both drums were vibrating, the vibration level was also generally slightly higher when the roller travelled in reverse.

After completing the first 15 normal passes with this roller, and recording the vibrations from a number of other movements, accelerometer arrays were attached to the two drums. This was the point at which it became apparent that the vibratory mechanism in the rear drum had not been functioning. The testing was repeated after having repaired the vibratory mechanism.

On the hoggin, the Ingersoll-Rand DD65 began to break the larger flints after only a few passes. Consequently, only five normal passes were made, but this was sufficient for the ppv at the foundation to show no further increase as the number of passes increased.

A.6.5 Ingersoll-Rand SD150

The Ingersoll-Rand SD150 had separate control settings for amplitude, frequency, travel speed and engine speed. All standard passes were undertaken at the maximum settings. Following the normal testings, vibrations were recorded from passes made at different control settings.

The normal passes were all undertaken with the roller travelling in the same direction to avoid any possible

effects associated with the travel direction. The data from the foundation level geophone show a progressive increase in ppv with pass number. Following ten passes on the hoggin, two reverse passes were recorded, with all the controls set identically. The reverse passes gave rise to much lower levels of vibration than did the forward passes.

The variable controls on this machine enabled the effect of different frequencies (and hence centrifugal forces) on the radiated vibration level to be investigated while all other parameters remained constant. This was the only machine tested which had this capability.

A.6.6 Bomag BW161AD-CV *Variomatic*

The Bomag BW161AD-CV tandem roller which was tested had the front drum fitted with Bomag's *Variomatic* system. The *Variomatic* system uses two counter-rotating eccentric shafts to generate vibrations which can be directed either vertically or horizontally, or at any angle in between, by altering the relative phase of the two eccentrics (Byles, 1997). The angle of vibration can be controlled manually from within the vehicle, or set to change automatically. The automatic mode uses accelerometers mounted on the drum to monitor the behaviour of the drum, which is related to the stiffness of the material being compacted. As the stiffness increases, the orientation of the vibration rotates to become directed increasingly towards the horizontal. This system is designed to eliminate the crushing of aggregate and the loosening of upper layers. However, on the fill materials used for the current research, sufficient stiffness was not achieved to make the compaction direction change automatically from the vertical.

Testing was carried out with the roller operating both as a single drum machine, with only the front drum vibrating, and as a tandem roller. For all of the normal pass tests, the vibration of the front drum was directed vertically. In addition to the tests undertaken for the other rollers, a series of measurements was made with the vibration orientation moved in 18° increments from vertical to horizontal. This included both travelling compaction and starting and stopping transients with the roller not travelling.

A.6.7 *Wacker DPU 6760 vibrating plate*

The experimental method used for the Wacker DPU 6760 vibrating plate compactor was the same as that used for the rollers. It was not possible to measure directly the vibration of the plate because the amplitude was too great for the accelerometer system.

This was the final piece of vibratory equipment tested during the main phase of the experiment. The following items of plant were tested as they became available through other research projects.

A.6.8 *Hamm 2422DS*

Three months after completion of the main trials, a Hamm 2422DS single drum pad foot roller became available. The test area had been exposed to the weather during this period and had become too wet to traffic. A stockpile of hoggin was still available so the wet upper layer of hoggin was removed and replaced. The clay was not usable so the roller could only be tested on the hoggin. Sixteen normal passes were made with this roller, all travelling forwards. Time did not allow for any further investigation to be undertaken.

A.6.9 *Rammax 2400*

A small Rammax 2400 pad foot tandem roller was available at the same time as the Hamm 2422DS. Only one forward pass was made on the hoggin since the fill had become too soft for further trafficking. As for the Hamm 2422DS, the clay could not be trafficked at all.

A.6.10 *Bomag BW120AD-3*

In addition, to the main experimental work, there arose the opportunity to undertake a further investigation into the effect of fill on the vibration level. A series of tests was undertaken on a section of road which, as a part of a separate research project, was under construction on a foundation consisting of nine different subgrade materials. Vibration was measured from the use of a Bomag BW120AD-3 on these different materials.

A.7 Further investigations using the Bomag BW161AD

The initial processing of the data from the main experimental phase of this research revealed three particular issues which required clarification. Consequently, further testing was undertaken using a Bomag BW161AD, as described below, to address these points.

Initial analysis revealed that, when plotted in log-log space as resultant ppv against distance, the data described a curve, rather than a straight line, as is conventionally assumed. A greater number of data points than were available from each individual roller were therefore required to define properly this curve.

This initial analysis had also demonstrated that, for most rollers, the resultant ppv at any distance was greater when the rollers operated on the clay than when they were operated on the hoggin.

The third issue to be investigated was the relation between travel speed and ppv. From the earlier data there was clearly a relation between these two parameters, but the amount of data available for any one roller was insufficient to define clearly the form of the relation. This Section discusses the approaches taken to investigate these issues.

There were considered to be two possible explanations for vibration measured from rollers operating on the clay being greater than when operating on the hoggin. One explanation might be that some interaction between the roller and the fill, or some characteristic of the fill, reduced the source vibration level such that the energy transmitted into the environment was lower for operations on the hoggin than on clay. Alternatively, the same amount of energy may have been radiated from both situations, but the energy transmitted into the hoggin was attenuated more because an additional acoustic impedance mismatch was encountered, which would reduce the energy reaching the geophones, since the clay was located between the hoggin and the geophones for all the measurements.

To investigate this, an additional set of tests were

undertaken during which measurements were made simultaneously on both sides of the test area while operating a Bomag BW161 tandem roller on the hoggin. It was not possible to operate the roller on the clay because it remained too soft to traffic. However, an experiment was devised to address the problem. Three triaxial arrays of geophones were positioned in a line extending away from the test area on the opposite side to that which had been used for the earlier testing, ie on the side closer to the hoggin. Additionally, two further arrays were positioned on the side of the test area closer to the clay. With this arrangement of geophones, if the acoustic impedance mismatch was the reason for the difference in the data, the vibration levels recorded on the side closer to the hoggin would be greater than those recorded on the more distant side at any given distance. The roller made 12 passes of the hoggin, with only the front drum vibrating to minimise any changes in vibration level which might occur due to increasing compaction of the fill. These passes were all performed at different speeds so that the data could also be used to determine a relation between speed and peak particle velocity (ppv).

A second set of measurements was undertaken to define the shape of the attenuation curve. The roller could not be operated on the clay and it was only possible to achieve a spread of geophones at distances of up to 100m on the side of the test area closer to the clay. The roller was therefore operated on the natural ground. The same geophone positions were retained for each set of measurements and three passes were made, each at a different distance from the end of the line of geophones, giving 15 different distances to define the shape of the curve between 1m and 121m.

A.8 Investigation of the effect of fill type on groundborne vibration level

The main experimental work had shown that, for most rollers, the resultant ppv at any distance was greater when the rollers operated on the clay than when they were operated on the hoggin. Further data would therefore be valuable to investigate the effect of the properties of the fill on the vibration level. A separate trial was therefore undertaken using a test facility being constructed for a different research project.

The project required construction of a 36m length of road, founded on contiguous sections of subgrades of chalk, London clay and a silty sand (Figure A7). Each subgrade material was placed in three separate bays, each at a different moisture content. Thus, this presented an opportunity to measure the vibration levels arising from a constant source of vibration operating on a number of different materials. The measurements were undertaken following the placement and compaction of the capping material.

Triaxial arrays of geophones were positioned alongside the test structure. Two geophones were positioned adjacent to each bay, all at approximately the same distance from the fill. There was limited space available so no attempt was made to assess attenuation effects; this experiment was designed to look solely at the effects of fill on source vibration levels. It was attempted therefore to measure vibration at the same distance from the roller operating on each fill. To minimise the impact on the experiment for which the test road was constructed, it was necessary to restrict the amount of compaction undertaken. Measurements were made adjacent to each bay at two locations, to improve the confidence in the data. Two passes of the roller were made over each bay, one on each side of the road. The restriction on trafficking precluded the measurement of vibration caused during the starting and stopping cycles since this would have formed depressions in the surface of the capping. It was not possible to conduct the experiment on two of the sand bays because they did not have sufficient strength to support the roller. Only one geophone was positioned for the third clay bay because the roller had to be stopped on this bay. This restricted the length on which the roller could be operated at constant speed.

The tests were conducted using a Bomag BW120AD-3 tandem roller operated with only the front drum vibrating. The use of only one vibrating drum limited the compaction caused by these tests and made the location of the vibration source distinct. If both drums of the roller had been vibrating, there would have been periods during which the drums were operating on adjacent bays containing different materials. The roller was operated at its maximum speed for compaction and always travelling in the same direction. The time taken to travel the length of each bay was recorded to confirm that the travel speed was constant.

Following the vibration measurements, the *in situ* characteristics of the materials in the bays were assessed using the Transport Research Laboratory Foundation
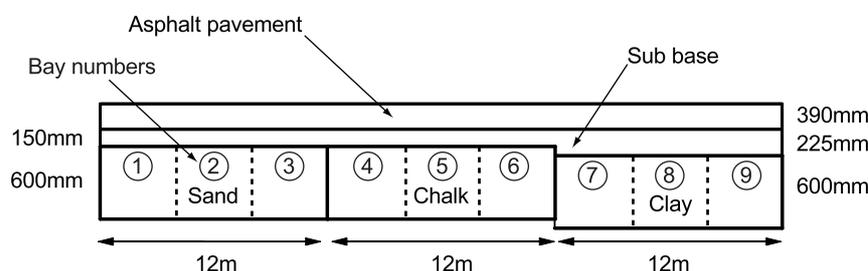

Figure A7 Section through trial road

Tester, the Falling Weight Deflectometer, the German Dynamic Plate Bearing Test and surface wave seismics (Matthews *et al*, 1996). These techniques all yielded stiffness values for the subgrade material which, although they differed in absolute values, showed similar trends in the relative stiffnesses of the materials.

A.9 References

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Abstract

The increasing size and power of construction plant and its potential to dissipate intrusive or possibly damaging levels of vibration into the environment, coupled with increasing attention being given to environmental aspects of road construction, have led to a need for improved methods of ground vibration prediction. While there is an increasing need to minimise the intrusive effects of construction works, over-conservative restrictions on vibration levels may lead to significant and unnecessary cost increases. This report provides data and advice against which objections to schemes may be judged and a means of assessing the environmental impact of vibration from road construction works. Predictors are proposed for vibratory compaction, vibratory piling and vibratory ground treatment, with three levels of the probability that the predictions will be exceeded. A predictor is also given for the likely upper bound vibration levels from impact piling in a range of ground types. Further predictors are given for vibration from dynamic compaction and tunnelling, and for groundborne noise from tunnelling. The appropriate British Standards for the assessment of the effects of the predicted vibration are reviewed and compared with other national standards.

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