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GROUNDBORNE VIBRATIONS GENERATED BY HGVs - EFFECTS OF VEHICLE, ROAD AND GROUND PARAMETERS

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INTRODUCTION

Heavy traffic can produce vibrations in buildings close to the road and disturb occupiers. This paper describes results from a programme of research aimed at quantifying the relationship between groundborne vibration at the foundations of buildings and various parameters of the vehicles, road and underlying soil.

Tests on the research track at TRRL with a wide range of heavy goods vehicles (HGVs) established the trends in peak vibration levels with size of road surface irregularity, speed and load. The results of these initial tests were described in a previous paper [1]. In order to generalize the results to other site conditions, the transfer functions for an impulsive force applied to the road surface and the resulting ground vibrations at various distances up to 50m were measured for a wide variety of soil conditions ranging from soft peat and alluvium, through clays and sands to rock. By determining the average effects of these factors it was possible to estimate the likely range of vertical peak particle velocities (PPVs) at the foundations of dwellings under a variety of site conditions. These predictions are for groundborne vibrations generated at road surface defects up to a few metres in length. The much smaller-scale roughness of the road surface also gives rise to vibration but this is relatively small and is unlikely to cause significant disturbance though it can be detected at large distances because, in this case, the road can act as an extended line source [2]. This paper is concerned only with vibration produced by large-scale irregularities.

TESTS WITH HGVs

Eight HGVs were tested over artificial humps and a depression on the TRRL research track. Six were articulated vehicles and were assembled using the same tractor unit, a Daf 3300 with two axles on steel leaf springs. The suspension systems for the trailers covered a wide range and included two and three axle bogies with steel and air suspension and a two axle rubber sprung system. The other two HGVs were rigid vehicles, one being a two axle flatbed lorry and the other a four axle tanker with twin steering and drive axles. Both had steel leaf springs.

The vehicles were tested at speeds in the range 16 - 80km/h both fully laden (ie close to the legal axle weight limits) and empty over a range of profiles. These were designed to represent a wide range of surface defects resulting from poorly backfilled holes and trenches on public roads. Figure 1 gives details of these profiles.

The particle velocities produced on the track surface were measured by triaxial geophone arrays fixed at 2 and 6m from the nearside wheel paths. Further details of the test protocol and recording and analysis systems are given in [1].

RESPONSE AND PROPAGATION IN DIFFERENT SOILS

Previous measurements by the author had indicated that the vibrations generated in soft ground such as alluvial and peat soils were much greater than was the case for firmer soils under broadly similar conditions [3,4]. It was, therefore, considered essential to make an adequate correction for ground conditions when extrapolating from results obtained on the research track where the subgrade is firm. The requirement was for the transfer function between the force input on the road and the resulting vibration to be established for representative soil types ranging from very soft to very firm. Once determined, these functions would allow PPVs to be calculated for a particular site by factoring the PPV expected on the research track by the ratio $|H_s(f)|/|H_t(f)|$, where $|H_s(f)|$ and $|H_t(f)|$ are the moduli of the transfer functions or mobilities at the site and on the track and 'f' is the forcing frequency. This frequency is typically between 10 - 12Hz and results from the "wheel hop" mode of vibration of the HGV suspension.

To determine the transfer functions, the Falling Weight Deflectometer (FWD), which was designed to measure road pavement characteristics, was used to produce very carefully controlled road surface impacts. The device consists of a trailer-mounted impact device with associated deflection and load measuring transducers and a system processor and micro-computer for controlling the equipment and recording measurements. The size of the impact is varied by altering the distance through which the moving mass falls or by changing the size of the mass itself. The peak loading was adjusted to between 60 - 70kN, which is of the same order of dynamic loading as that produced by an HGV suspension when passing over a significant surface irregularity. The FWD's electronics were adapted to enable the time history of the dynamic force at the road surface to be recorded. Figure 2(a) and (b) shows typical time histories of force and resulting ground vibration from the first impact of the falling weight. Secondary impacts occurred due to rebounds, but the signals produced were not used in the computation of the transfer functions except in a small number of cases where it was impossible to identify the velocity trace from the first impact due to the broadening with distance of the wave packets from each impact.

Measurements were made at 13 sites on both bituminous and concrete road surfaces. Sites were chosen on a range of soils from very soft to hard and these included peat, alluvium, London and Boulder clays, sands and gravels, and chalk rock. The portion of the test track where testing took place was on sand and gravel deposits. Triaxial geophone arrays were placed just beneath the surface of the ground at distances of 3,6,12,25 and 50m from the FWD where possible and simultaneous recordings of dynamic force and particle velocity were made on an instrumentation tape recorder. The aspect of interest here, of course, is the extent to which traffic vibrations are transmitted into buildings, rather than the response of the ground. Consequently the transfer functions of building foundations of private, mainly two storey dwellings, were also determined on most soils. These were compared with the values expected in the unloaded soils at the same distance from the source so that scaling factors could be determined relating the two conditions. These factors were then used to predict PPVs at building foundations from the results obtained from measurements in unloaded soils.

The data was processed using a two channel digital signal analyzer. Figure 3 (a) and (b) show the modulus of the transfer function in alluvium at 6m and the associated coherence function based on four impacts. The coherence function is a measure of the signal strength in the output (velocity) that is due to the measured input (force) at each frequency. The coherence is close to unity at the frequencies of interest indicating insignificant measurement noise errors in the determination of the transfer function. This was also the case for the other measurements.

RESULTS AND ANALYSIS

The aim of the analysis was to develop a practical prediction method for the maximum expected PPV at a building foundation, using measures that would be readily available. The effects of different suspension systems will not be discussed in this paper, since this type of information would not normally be available. Different vehicle types do produce significantly different amounts of vibration, but it will be seen that these differences are minor compared with the very large effects of ground conditions. Therefore the results for the range of HGVs were used to establish the main trends without unduly affecting the overall accuracy of predictions. It was also decided at an early stage to restrict the method to the prediction of the vertical component of vibration, as this is the dominant component of traffic vibration at building foundations. Previous studies on a variety of soils indicate how other components of vibration in different parts of buildings are likely to be related to vertical vibrations at the foundations [3,4].

In Figure 4 the vertical PPVs at 6m produced by each fully laden vehicle travelling over the profiles (including the track surface which varied by +/-7mm) at 48km/h are plotted against the maximum height or depth. It can be seen that this dimension of the profile is a reasonable indicator of likely PPV despite the wide variety of profile shapes. Linear regression analysis showed that the average PPV for a 25mm high profile was 0.7mm/s, the slope being 0.028mm/s per mm increase in height or depth. For unladen HGVs at 48 km/h, the average PPV is lower by 0.08mm/s. Since in practice many vehicles passing a site will be laden, predictions should be based on the average response of fully laden lorries.

Figure 5 shows how PPV varies with speed for laden trucks over a profile of 0.6m x 25mm. The trend is a linear increase with speed and this is the case for other profiles. For particular vehicles some combinations of speed, profile and load produced variations from these linear relationships [1]. To account for these interactions in the predictive model would lead to unacceptable complications in what is intended to be a practical prediction tool. In many cases on public roads, the irregularity is significant in only one wheel path and this is usually on the nearside. Tests have shown that in this situation PPVs are generally significantly lower than the values recorded when identical profiles are in both paths. The factor 0.75 was considered to be appropriate for scaling predicted values in these cases.

Figure 6 shows how the moduli of the transfer functions at 12Hz vary with distance for six different ground conditions. The figure clearly shows that there is an enormous difference of over two orders of magnitude between the responses of a soft soil (peat) and very hard ground (chalk rock). Type of road construction (ie whether bituminous or concrete) appeared to have relatively little effect on transfer functions. The dependence on distance seen in Figure 6 is very broadly approximated by r^{-1}, where r is the distance from the source. Established theoretical work has shown that when an oscillating point load is applied to an elastic half space the dominant motion in the far field is a surface (Rayleigh) wave, the vertical component of which decreases with distance as $r^{-½}$ [5]. In practice it has been found that there are differences between observed and theoretical attenuations and to reduce this discrepancy the absorption of wave energy by the soil has been included in the theoretical calculation of the attenuation function [6]. If it is assumed that the absorption of this energy is proportional to the amount of energy entering a given soil layer and its thickness then the amplitude can be shown to vary with distance from the source as $r^{-½}\exp(-\alpha r)$. The absorption coefficient should increase as the square of the frequency if the soil behaves as a simple viscoelastic system in which dissipative forces are proportional to the velocities of deformation [6]. Attempts were made to fit this function to the data obtained in the present study but agreement was generally poor at the largest distances. Some other studies have also found significant discrepancies [7]. However when linear regression lines were fitted to the logarithmic transform of the data at 12Hz for each site, it was found that the correlation coefficients ranged from 0.922 to 0.998 indicating good agreement with an attenuation model based on a simple power law r^x, where x is the power coefficient. As Table 1 shows, x depends on soil type, but is fairly close to -1.

It is clear from Table 1 that there is a range of power coefficients even for similar soil types. The relatively low attenuation rates found in sands are consistent with previous work which has shown that absorptive losses are generally small [7]. The best estimates for predictive purposes are the average values most appropriate to the soil type.

Table 1 also lists the range and average values of the moduli of the transfer functions at 6m for the six ground conditions together with the corresponding values for building foundations. For all soils except peat, average values at foundations were similar to or just below those on unloaded ground and, therefore, it was prudent to assume that foundation and ground responses were identical when making predictions. In the case of peat, foundation response was found to be significantly less and a scaling factor of 0.22 has been applied. In addition, for this soil the transfer function at 10Hz was found to be much greater than at 12Hz and so the value for the lower frequency has been used. The relatively low resonant frequency of the building in the vertical direction on this very soft soil may be responsible for this effect.

PREDICTIONS OF VERTICAL PPV AT BUILDING FOUNDATIONS

The method is based on making predictions of vertical PPV at 6m using the trends with profile amplitude and speed based on the results obtained from tests described above involving a wide range of HGVs. The result is then scaled by the most appropriate ground scaling factor. By combining these factors, the expected value of maximum vertical PPV at a building foundation can be calculated as:-

$$0.028a.(v/48).t.p.(r/6)^x$$

Where a = maximum height or depth of surface defect in mm, v = maximum speed of HGVs in km/h and t = ground scaling factor (see Table 1). If the surface defect occurs in one wheel path only then p = 0.75, otherwise p = 1. The distance of the foundation from the defect in metres = r and x = power factor, which can be obtained from Table 1 for the most appropriate soil type.

To check the accuracy of the formula, predictions were made at 8 sites where long term measurements had been made at building foundations. The measured values were obtained by sampling hourly for 15 min between the hours 0900 and 1800 [3,4]. Figure 7 shows the measured and predicted maximum PPV at these sites. The data from all but one site lie reasonably close to the correct prediction line. The outlier is for a site thought to be on soft alluvium although the measured value suggests a ground scaling factor for a firmer soil would be more appropriate. Another possibility is that the measured profile was too close to the kerb (the road surface defect in this case being a sunken drain cover in the gutter) and that during the measurement period no HGV wheels passed completely over the irregularity.

DISCUSSION AND CONCLUSIONS

The significant factors affecting the generation and propagation of groundborne vibration have been quantified and predicted results for the maximum vertical PPV are generally in reasonable agreement with the limited number of measurements. Ground conditions have been shown to have a highly significant effect on vibration levels and the largest errors in making predictions are likely to be associated with the choice of the appropriate ground scaling factor. For this reason it is suggested that, when predictions are being made, use should be made of relevant borehole logs to identify ground conditions. For measurements in this country the British Geological Survey can be consulted. Since a range of attenuation rates were found even for soils of similar type, there are likely to be significant prediction errors at relatively large distances.

Generally predictions are likely to be most accurate where the profile is a few tens of millimetres high and speeds are below 80km/h since most measurements were made for this range of conditions.

Clearly a larger sample of cases is needed to fully validate the prediction method and to quantify errors more precisely. However the prediction method should be useful initially in determining whether there is likely to be a ground vibration problem arising from road surface defects and the possible scale of the effects. If the predicted maximum PPV at foundation level is significantly in excess of 0.3mm/s (the threshold of perception for continuous sinusoidal vibration [8])then disturbance to occupiers and even complaints may be expected.

ACKNOWLEDGEMENTS

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GROUNDBORNE VIBRATIONS GENERATED BY HGVs

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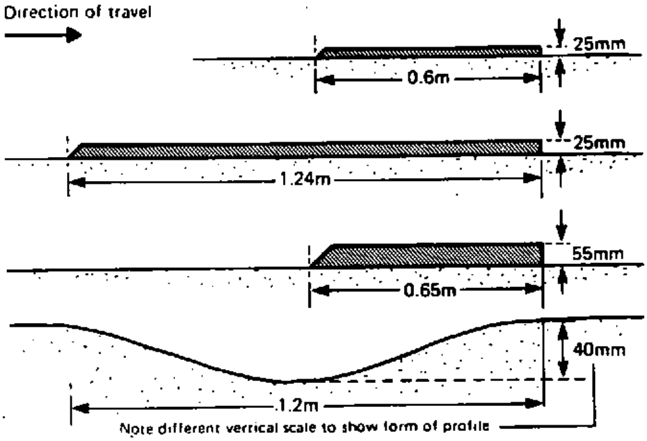
Ground type: Description	Ground type: Condition (if known)	Number of sites tested	Power coefficient for attenuation with distance 'x' **: Range	Power coefficient: Average	Modulus of transfer function (x 10^3) in mm/s per kN, In ground at 6m: Range	In ground at 6m: Average	Expected value at foundations 6m	Ground*** scaling factor $t=\frac{\lvert H_s\rvert}{\lvert H_t\rvert}$
Peat *)	Soft	1	-	-1.19	-	189	41.9	3.84
Alluvium)		2	-0.79 to -0.80	-0.79	72.5 to 82.0	77.3	77.3	7.07
London clay		3	-0.99 to -1.13	-1.06	20.9 to 56.3	33.8	33.8	3.10
Sand/gravel		3	-0.69 to -0.82	-0.74	9.92 to 11.0	10.3	10.3	0.94
Boulder clay		3	-0.71 to -1.18	-0.93	2.43 to 6.67	4.73	4.73	0.43
Chalk rock		1	-	-1.08	-	1.14	1.14	0.10

* For peat soil transfer function values and power coefficient are for 10 Hz

** Power law for attenuation with distance is r^x where r is distance from source

*** $|H_s|$ and $|H_t|$ are the moduli of the site and track transfer functions

Table 1. Effect of ground characteristics on transmission of vibration



Note: The track surface had a level surface to within ± 7mm within 5 metres of the mid profile position

Fig.1 Test profiles

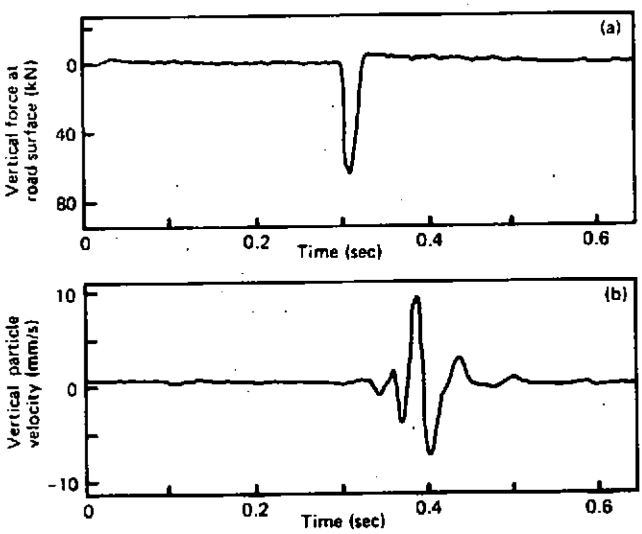


Fig.2 Force pulse from a falling weight deflectometer and resulting particle velocity at 6m in alluvial soil

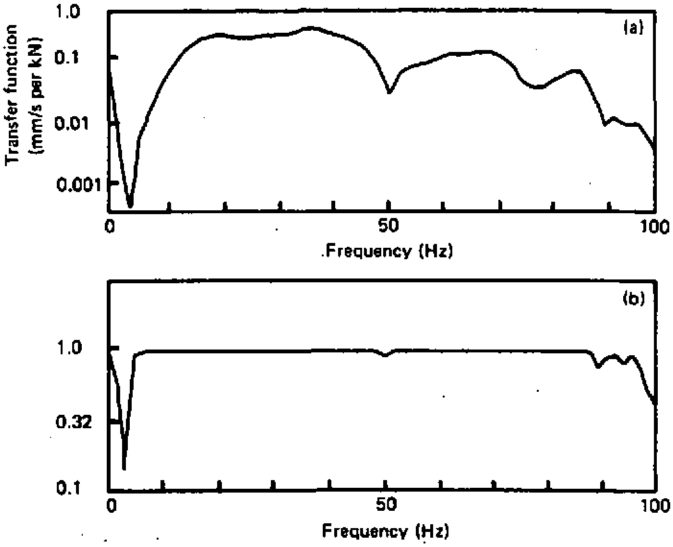


Fig.3 Transfer function and associated coherence determined at a distance of 6m in alluvial soil using the falling weight deflectometer

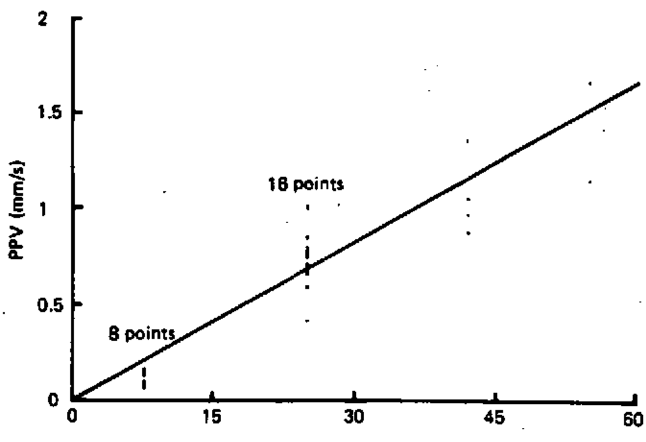


Fig.4 Vertical PPV at 6m with profile height or depth at 48 km/h. – TRRL track

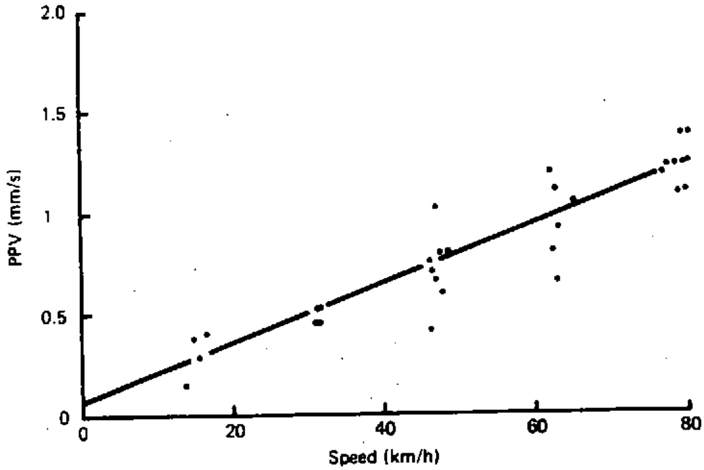


Fig.5 Vertical PPV at 6m with vehicle speed over 0.6m x 25mm hump – TRRL track

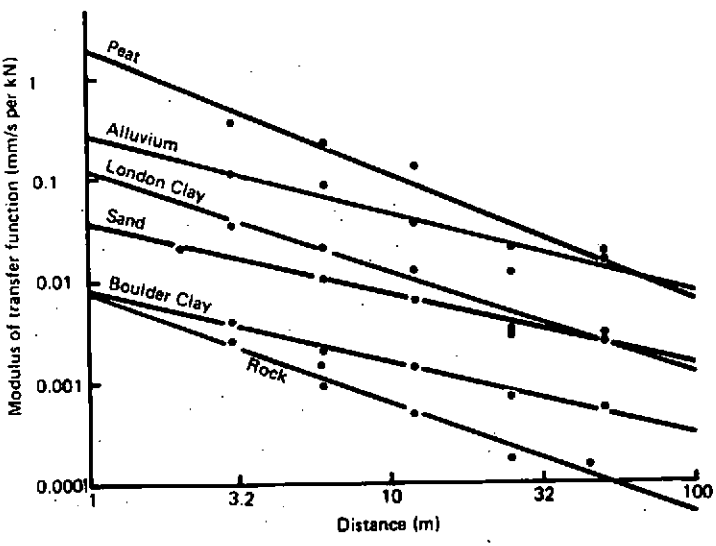


Fig.6 Transfer function for vertical PPV at 12 Hz by distance

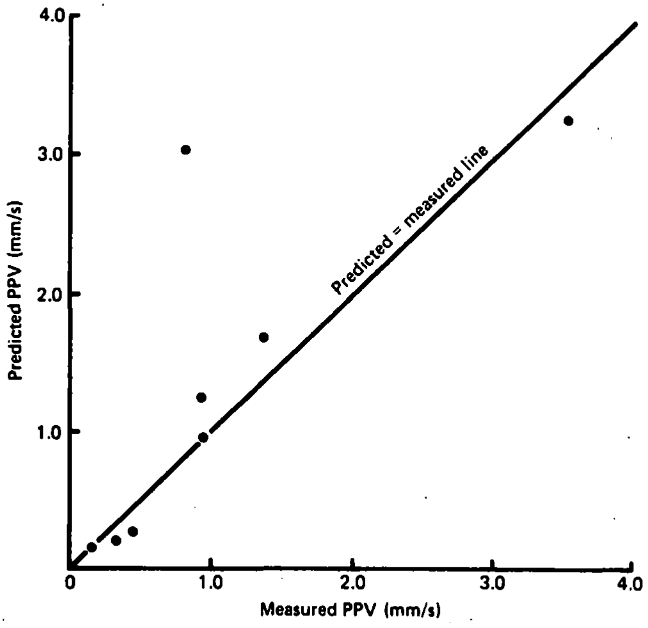


Fig.7 Measured and predicted maximum vertical PPV's at building foundations

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THE WAYSIDE VIBRATION FROM TRAINS

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Railway vibration is a subject which has had relatively little coverage in the literature. Most published information relates to buildings very close to urban rapid transit systems which are often underground. In these situations vibrations tend to be at a higher frequency than those caused by the overground railway and problems are often caused by re-radiated noise as much as by perceptible vibration. The conclusion which could be drawn from this lack of published information is that vibrations from the overground railway are not a major problem. It is British Railways' experience that this is the case in that only a small percentage of people living next to a railway line are concerned at the vibration produced. However, at certain locations widespread concern is expressed at the level of vibration from particular train types.

Written complaints of vibration from running trains are usually investigated by British Railways Area Civil Engineers. A visit is made to the complainant and a report compiled of all railway characteristics and details of the complaint. From these reports common reasons for complaints can be found and further investigation undertaken by the Research Department.

It is the purpose of this paper to look at some of the factors which influence the level of vibration at a particular site.

GROUND TYPE

The question "how much vibration does a certain train type make?" is frequently asked. Unfortunately there is no simple answer since the ground conditions have a strong influence on the vibration produced. The noise from a railway is readily predictable with a few qualifications because the air is a simpler and more uniform propagation medium than the ground. The ground moduli and strata interface depths will vary from site to site and lead to different vibration spectra from the same train type. Figure 1 shows the extremes in vibration which have been measured on the ground surface for nominally the same railway conditions. The trains were both loaded coal trains made up of the same type of wagon and running on good quality welded track at approximately the same speed. The vertical vibration is quoted since this was higher than the horizontal in both these cases. We have found this is the normal situation but at a few sites larger horizontal vibrations have been observed.

It is interesting to note in these plots that the same frequency peaks occur at both sites but the amplitude ratio of each varies markedly. The peak at 25Hz coincides with the passing frequency of the discrete support points of the rail by the sleepers at regular intervals of 0.7m. At site A this is a minor source of of disturbance whereas at site B it causes the dominant vibration.

In an attempt to explain these wide differences a two-dimensional analytical model developed at Southampton University [1] has been used. Figure 2 shows the predicted amplitudes at 25m from a constant excitation force at frequencies up to 64Hz. Two models have been used with the same ground moduli in each. The first is that of a strip load on a uniform halfspace and the second is for a strip load on a finite layer of ground lying over a rigid bedrock. The halfspace allows all frequencies to propagate but the layered grounds have a low frequency limit below which waves do not propagate. This limit is approximately where the depth of layer is half the shear wavelength.

Figure 3 shows the influence of the shear wave velocity on the level of vibration produced for cases where the layer depths have been chosen to be half the shear wavelength at about 35Hz in each case.

These models help to explain some of the site differences and may account for the observation at one site where a wheel passage frequency of 3Hz propagated with little attenuation out to 80m. For this to occur the ground would have to be a uniform and deep layer.

VEHICLE TYPES

A major distinction which can be made is between passenger and freight vehicles. A passenger vehicle has a lighter axle load than a loaded freight vehicle and a complex suspension design to give satisfactory passenger comfort at high speeds. A freight vehicle can have axle loads up to 25t and although its suspension design is simpler it often needs to cater for a wider load range (6t tare to 25t laden) and ensure a safe and stable ride.

Passenger train spectra tend not to have the dominant peaks seen in freight train spectra and rarely is the vibration perceptible at 25m even from high speed trains.

The exception to this the case of some electric multiple units running on jointed track where perceptible vibrations occur owing to the axle hung motors giving a high unsprung mass.

Differences in the suspension design of the heavy freight vehicles result in different levels of vibration being caused. New designs with more controllable levels of suspension damping which can be maintained over the service life of the vehicle are expected to to minimise the vibration produced.

SPEED

There is a natural tendency for complainants to feel that excessive vibration is caused by "speeding". This is reinforced by the fact that often it is not all the passing trains which give rise to high vibrations. However, it has been found at some sites that there is a critical speed, somewhere below the maximum operating speed where the vibration is as high as at the maximum speed. In these cases it is more likely that the train thought to be speeding is merely travelling at this critical speed. This effect was first reported in the 3rd International Railway Noise and Vibration Workshop [2].

A recent example of this occurred on a freight only line where the maximum operating speed was 60km/h. An experiment was carried out using a test train on the single line, continuously welded rail, track at a range of speeds. Measurements were taken on the outside wall at ground level and the centre of an upstairs flat of a two storey building 50m from the railway. Figure 4 shows the experimental results together with measurements for service trains for the floor vibration. A correction factor of 1.4 has been applied to the test train results to make them comparable to the longer service trains on account of the different no of vehicles involved.

The line on Figure 4 is drawn as vibration proportional to train speed. At most speeds the vibration levels are close to this line with the exception of 56-64km/h where higher levels of vibration occurred. It was possible at this site to run the trains at a slightly slower speed without causing operating difficulties while achieving a significant reduction in vibration. If the operating speed had been nearer to the maximum of 96km/h, an unacceptably large change in speed would have been required to produce a worthwhile change in vibration.

Experiments at some sites with different vehicles have not shown the vibration to peak at a critical speed and in these cases the increase in vibration is proportional to speed.

HOUSE RESPONSE

Most of our measurements have been taken on the ground surface with the loose top soil removed. Since it is necessary to know the level of vibration in dwellings, comparisons have also been made between open site measurements and the lower wall of buildings in the same area. For vertical vibration at low frequencies, the ground and building vibrations are essentially equal but at higher frequencies ground vibration levels are higher. The actual frequency at which the ground and building levels begin to differ does vary from site to site but we have found it to lie in the range 12-25Hz.

Floor resonances do increase the level of vertical vibration but we have found these to lie in the range 25-40Hz in conventional dwellings and the increase in the total vibration for the low frequency spectra is only slight.

Most high levels of railway vibration are at the lower frequencies and in these cases measurements taken on the ground will be approximately the same as those to which people are exposed in buildings.

RATING OF RAILWAY VIBRATION

Some people are concerned about any perceptible vibration from trains. It is thought that the reason for this might be the fear that damage is being caused to their homes. British Railways have a policy of visiting all complainants to assess the reasons for the complaint and where necessary the opportunity is taken to explain that the available evidence is that railway vibration is not at a level which is likely cause damage.

The method of rating vibration we use is based on the rmq dose described in BS 6472. The "satisfactory" continuous vertical vibration for frequencies above 8Hz corresponds to 0.2-0.4mm/s during the daytime and 0.14mm/s at night.

This is converted to a total dose for the time period considered using:

For daytime: Total dose = $\sqrt[4]{(0.2)^4 \times 16 \times 3600}$

$= 3.098$

For night time Total dose = $\sqrt[4]{(0.14)^4 \times 8 \times 3600}$

$= 1.824$

The total dose for the train service is calculated from

$$\text{Total dose for train service} = \sqrt[4]{\sum_{i=1}^{i=n} N_i (v_i)^4 t_i}$$

Where N_i is the number of trains of type i

v_i is the rms vibration velocity for trains of type i

t_i is the passing time in seconds for trains of type i

The total train dose is then compared with the total satisfactory dose defined in the Standard.

The concept of dose provides a practical way of assessing a mixture of different traffic types and can be used to assess the changes in traffic patterns. Dose based on the vibration squared (long term rms) results in single events being allowed to produce unacceptably high levels while remaining within a long term rms limit. The fourth power relationship (long term rmq) appears to give realistic levels for both a single trains during the day and frequent train service. Although the specification provides a useful method in assessing vibration it must be remembered that there are many factors which influence a person's reaction to passing trains.

CONCLUSION

The prediction of railway vibration is not as straightforward as railway noise. Ground conditions play an important part in determining the level of vibration from trains at a particular site. For this reason care must be taken in applying data obtained at one location to any other.

There are some general conclusions which can be made. Passenger trains produce less vibration than heavy freight trains and are usually not perceptible at 25m even at high speed.

THE WAYSIDE VIBRATION FROM TRAINS

Although vibration generally increases in proportion to speed, at some sites a critical speed has been found in the lower speed range where the vibration from a freight train reaches a peak.

Railway vibration can be assessed using the rmq dose method described in BS 6472.

ACKNOWLEDGMENTS

Thanks are due to both the Director of Research and the Director of Civil Engineering of thr British Railways Board for allowing this information to be presented.

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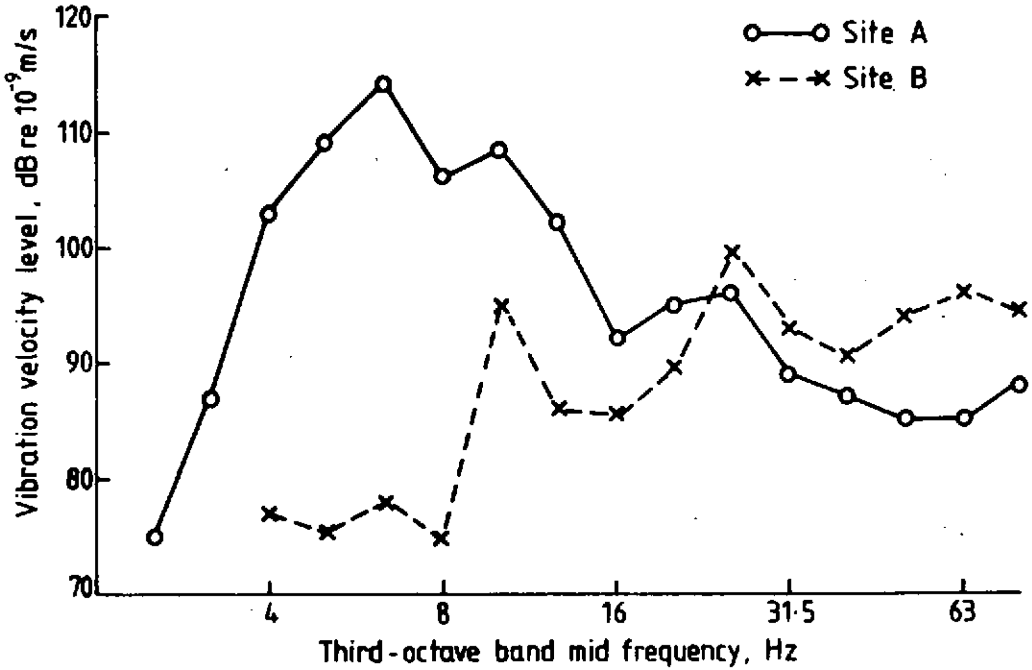

FIG. 1 VERTICAL GROUND VIBRATION SPECTRA AT 25m FROM COAL TRAINS AT 65 km/h

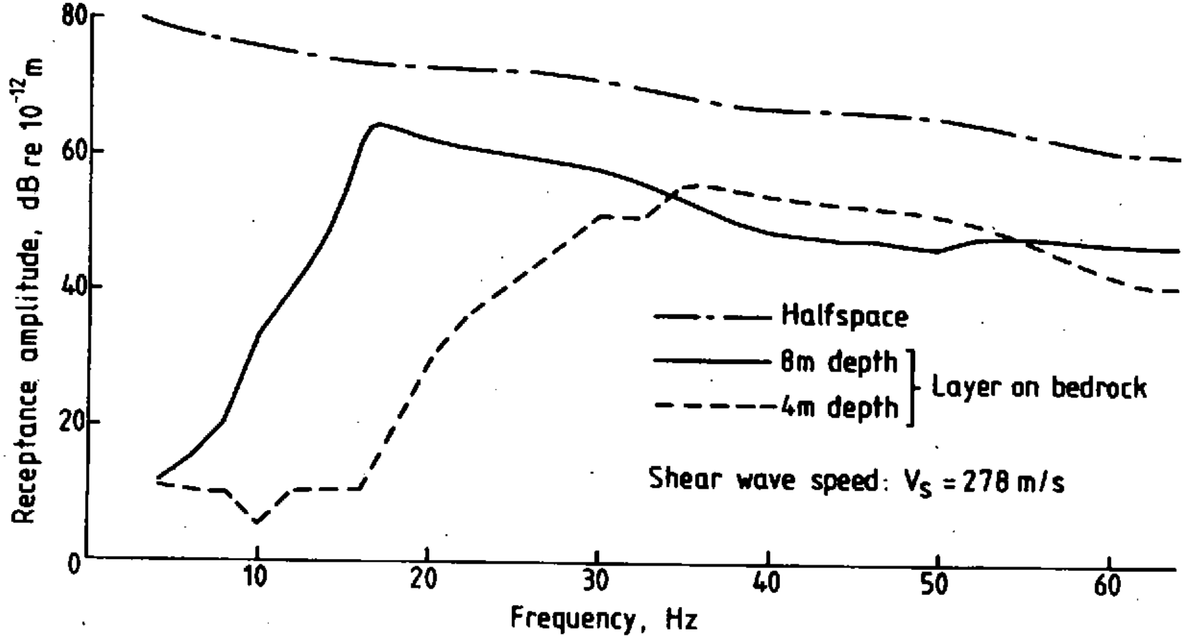

FIG. 2 PREDICTIONS OF RECEPTANCE AT 25m BY HALFSPACE AND LAYERED GROUND MODELS

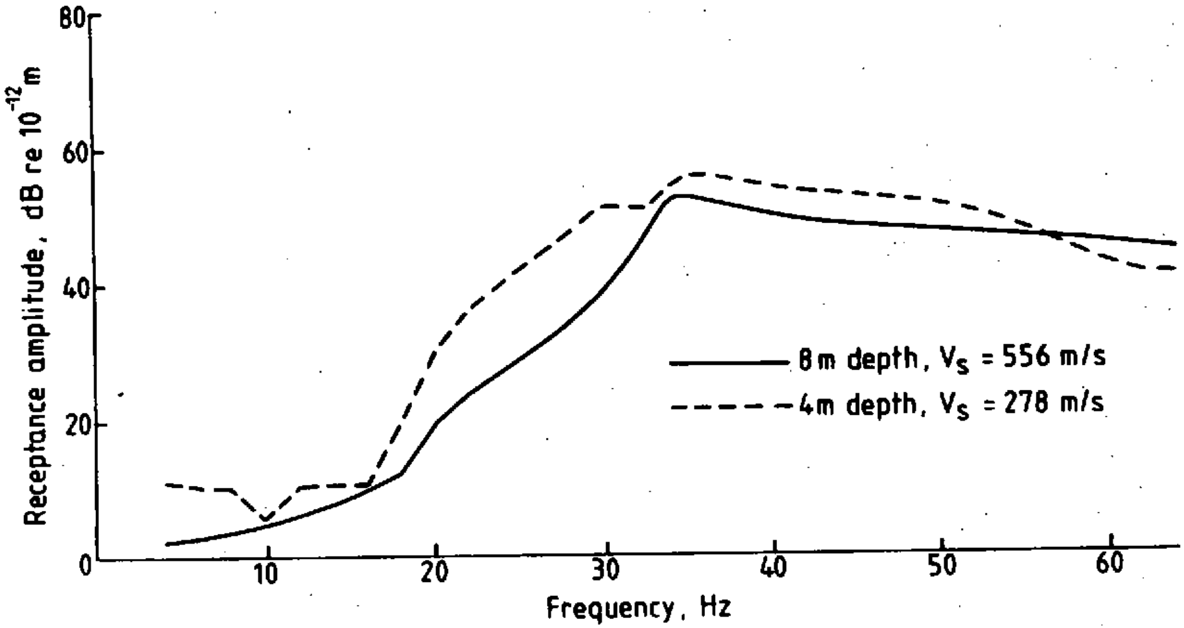

FIG. 3 PREDICTIONS OF RECEPTANCE AT 25m BY LAYERED GROUND MODELS

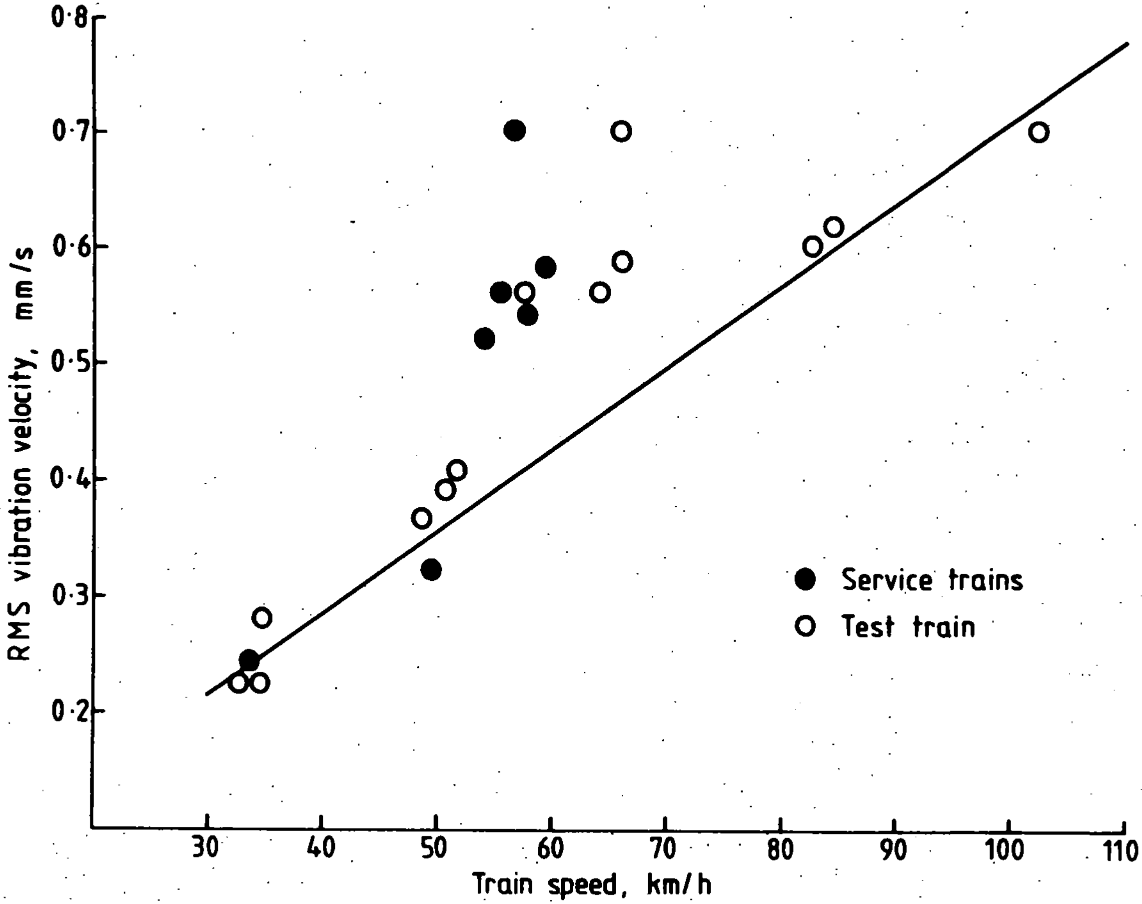

FIG. 4 VARIATION IN HOUSE FLOOR VIBRATION WITH TRAIN SPEED