Ground vibration from road construction
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An important note for the reader

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Abbreviations and acronyms

BS	British Standard
DIN	Deutsches Institut fur Normung (German Standard)
FTA	Federal Transit Administration
FWD	falling weight deflectometer
GPS	global positioning system
ICE	international construction equipment
IRI	International Roughness Index
NAASRA	The National Association of Australian State Road Authorities
NZTA	New Zealand Transport Agency
PPV	peak particle velocity
SCRCA	Swiss Consultants for Road Construction Association
SD	scaled distance
SRSS	square root of the sum of squares
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory
VDV	vibration dose value

Contents

Executive summary

In 2005, a programme of research was commenced to quantify levels of ground vibrations generated during various road construction activities and their attenuation with distance for different New Zealand soil types. The principal objective of the research was to develop a desk-based methodology, validated for New Zealand conditions, for determining separation distances from construction activity to ensure induced ground vibrations would have minimal effect on structures and their occupants. Ideally this methodology would make use of readily available data.

The research involved the following key elements:

1 A comprehensive review of international literature, which concentrated on the fundamental theory of ground-borne vibrations; measured vibration levels for different types of construction equipment; criteria for assessing vibration levels; and predictive methods.

2 Acquisition and analysis of ground vibration data from construction sites throughout New Zealand for representative mechanised construction equipment operating on a range of soil types. The specific equipment monitored comprised three dozers, 12 rollers, one grader, one stabiliser, two impact pile drivers and two vibratory pile drivers.

3 Investigation of the possible application of data acquired from commonly used geotechnical methods, notably scala penetrometer for estimating soil attenuation and falling weight deflectometer (FWD) to generate site-specific predictor curves for impact-related construction activity.

The principal conclusions arising from the research are as follows:

- Methods for predicting ground vibrations, unlike physical measurements, cannot characterise the vibrations in terms of their individual frequency components (ie frequency spectrum). Therefore, their output is limited to the amplitude of the vibration, which typically is in terms of peak particle velocity (PPV). From a consideration of various vibration-related standards, the following two criteria are suggested for preliminary assessment of predicted ground vibrations:
  - 0.5mm/s PPV for disturbance of building occupants
  - 5mm/s PPV for building damage.
- The above criteria do not apply to soils susceptible to vibratory densification or liquefaction. In general, these soils comprise uniform grain size (ie low cohesion) silts below the water table and have a very low scala count, ie about two blows per 50mm. For such cases, specialist geotechnical advice should be sought on the vibration levels that could cause consolidation or densification of the soil, which would result in differential settlement and consequent building damage.
- Ground vibrations measured during site preparation activities on representative New Zealand soil types were found to span a wide range of magnitude from 0.4mm/s PPV to 11mm/s PPV when standardised for a distance of 10m from the vibration source.
- No significant correlation between weight of the equipment and the resulting ground vibration was observed, with the heaviest piece of machinery, a 33 tonne static roller, inducing one of the lowest measured ground vibration levels (0.7mm/s PPV at a distance of 10m).
- There could be a significant difference in the level of ground vibration generated depending on the make and model of the equipment.
- Contrary to expectation, softer soils did not always produce the larger magnitude vibrations.

- Pile driving and vibratory compaction generated the most severe vibrations.
- Piles driven to refusal generated larger magnitude ground vibrations than piles just driven through the soil. Refusal is the condition when the effective energy of the hammer is no longer sufficient to cause penetration (the hammer is too light or velocity at impact too little).
- Vibratory piling, which is the vibrating of piles into the ground at fixed frequency, provided a significant reduction in vibration magnitude (quarter to half) over impact piling, particularly in easily mobilised soils, such as well sorted sand.
- Measured vibration levels from impact pile driving and vibratory pile driving agreed well with estimated vibration levels using empirical formulae given in appendix E of British Standard BS 5228-2:2009 *Code of practice for noise and vibration control on construction and open sites – part 2: Vibration.*
- Soil attenuation characteristics determined the rate at which ground-borne vibrations decayed with distance from source. It was observed that vibrations of higher frequency decayed faster than those of lower frequency and so any modelling of soil attenuation should take into account the dominant frequency of the vibration source.
- Soil attenuation properties derived for main soil types found in New Zealand are summarised in the table below:

Description of soil	Frequency independent soil property, ρ (s/m)	Attenuation coefficient, α, at 5 Hz(m^{-1})
Silty sand	2.04×10^{-4} – 3.18×10^{-4}	0.003 – 0.005
Sandstone	1.47×10^{-4} – 6.12×10^{-4}	0.002 – 0.01
Pumice fill	8.22×10^{-4} – 1.23×10^{-3}	0.013 – 0.019
Peat	8.30×10^{-4} – 1.94×10^{-3}	0.013 – 0.03

- The frequency independent material property of soil, ρ (s/m), is related to the frequency dependent attenuation coefficient, α (m^{-1}) by the following relation:

$$\alpha = \rho \pi f \text{ where } f = \text{frequency of source vibration } (Hz)$$

- Both average and maximum scala penetrometer readings were shown to be very good predictors of soil attenuation, even for volcanic soils, with a coefficient of determination (r^2) of about 0.9 being achieved. However, because the maximum scala reading may just relate to a narrow band and not be representative of the soil material, the equation below should be used to obtain estimates of site-specific soil attenuation coefficient:

$$\alpha(5Hz) = 0.0364 e^{-0.411 SCALA_{avg}}$$

where: α(5Hz) = soil attenuation coefficient (m^{-1}) for a frequency of 5Hz

$SCALA_{avg}$ = average scala reading (blows/100mm)

- Displacement time histories obtained during FWD measurements can be used to generate site-specific predictor curves for estimating vibration levels from impact-related construction activities such as compaction and piling. This method makes use of a square-root scaling law of ground motion and does not require detailed knowledge of the layering of the pavement structure or the geology of the surrounding site.
- Three methods are proposed for quantifying vibration impacts from construction activity:

1 Previously measured vibration source levels in combination with site scala penetrometer results. This method is suggested for equipment associated with site preparation. The vibration level at distance R_2 from the source of interest is calculated from:

$$V_2 = V_1 \left(\frac{R_1}{R_2}\right)^{0.5} e^{-\alpha(f)\times(R_2-R_1)}$$

where: V_1 = the measured peak particle velocity at distance R_1 (m)

V_2 = the peak particle velocity at distance R_2 (m) from source

$\alpha(f)$ = soil coefficient for the dominant frequency f (Hz)

2 Empirical formulae given in BS 5228-2:2009. This method is suggested for vibratory compaction, impact and vibratory piling, dynamic compaction, the vibration of stone columns and tunnel boring operations.

3 FWD derived predictor curves. This method is suggested for impact-related construction activities, such as compaction and piling, wherever FWD data is already available.

- On the basis of the field measurements, the following indicative separation distances are required to ensure ground vibrations generated from construction activity are below the damage threshold of 5mm/s PPV:
  - >20m for general site preparation works
  - >30m for vibratory piling
  - >70m for impact/percussive piling.

The four recommendations arising from this research are:

1 The relationship between scala penetrometer readings and soil attenuation needs to be validated over a wider range of New Zealand soil types before it can be applied with confidence.

2 Similarly, the FWD method for generating site-specific predictor curves has not been validated under New Zealand conditions. Therefore, whenever the opportunity arises, the method should be trialled.

3 The NZTA already has a comprehensive coverage of FWD measurements for the state highway network. These measurements can be accessed through the FWD table in the road assessment and management (RAMM) database. In order to permit ready derivation of project-specific ground motion predictor curves from the FWD time histories, investigations should be undertaken to establish if peak velocities at each of the sensor locations could be retrospectively generated for the FWD data already in RAMM and for future FWD surveys if peak velocities could be recorded in addition to the peak displacements.

4 Over time, it would be beneficial to build up a vibration-risk map of New Zealand that would have as its basis measured soil attenuation coefficients. Therefore, when conducting ground vibration measurements for assessment purposes, it would be desirable to describe soils in a consistent manner to reduce any subjective nature and variability in the reporting. The method of soil and rock description contained in the NZ Geotechnical Society's (2005) guideline for the field classification and description of soil and rock for engineering purposes appears to be the most appropriate for field use and so is recommended.

Abstract

There is an increasing requirement to control and manage ground vibrations generated by road construction and maintenance activities through project specific construction management plans. The objective is to minimise any potential adverse effects. The ability to reliably estimate vibration levels of specific construction activities at the project planning stage and to assess their likely effect on structures and their occupants is therefore required. Typical vibration characteristics for various activities, including site preparation, dynamic compaction and piling were measured for representative equipment and soil types to obtain baseline values for use in preconstruction assessments and to enable validation of available prediction methods. A review of international standards was also undertaken leading to two proposed criteria against which predicted vibrations can be assessed for damage and human perception. The possible application of data acquired from commonly used geotechnical methods, notably scala penetrometer for estimating soil attenuation and falling weight deflectometer to generate site-specific predictor curves for impact-related construction activity, was additionally investigated. This led to the recommendation of three methods, which make use of readily available data, for estimating vibration levels from construction activity at any specified distance from the vibration source.

1 Introduction

Many construction activities related to road infrastructure give rise to ground-borne vibration that may cause damage to structures or be perceptible to occupants in adjacent buildings and, therefore, give rise to complaints. At present, there is a lack of consistency in where, how, why and when an assessment of likely ground-borne vibration levels is made apart from new state highways where it has been a contractual requirement for a number of years (part of *Minimum standard Z/19 – social and environmental management* (SM030 Z/19)) to assess potential vibration effects. This need to avoid or reduce as far as is practicable the disturbance to communities from ground-borne vibrations generated during construction, maintenance and operation of state highways is also covered in section 2.12 of the Transit NZ (2008) (now NZ Transport Agency) *Environmental plan*.

The absence of an assessment can be due to cost, if vibration measurements have to be made onsite, or a lack of knowledge on where to source such an assessment. This is an unfortunate state of affairs because if unexpected disturbances arise during construction, it becomes far more difficult to resolve any problems for affected building occupants if they have not been forewarned of the timing and likely impact of the disturbances.

As part of the resource consent process, there is also an increasing requirement for road controlling authorities and their consultants to establish whether or not vibrations generated during road construction will be problematic to structures and buildings and occupants of these buildings so that management of such vibrations can be specifically addressed in the construction management plan.

This report presents the findings of a research project which commenced in 2005 and aimed to quantify levels of ground vibrations generated during the various road construction activities and their attenuation with distance for different New Zealand soil types. The intended outcome of the research was the development of a desk-based methodology, validated for New Zealand conditions, that would allow the estimation of low-risk operating distances for particular equipment/construction activities. Ideally, the methodology would make use of readily available data.

The key objectives of the research were to:

- acquire sufficient information on measured ground-borne vibration levels arising from various forms of road construction operations through literature reviews, limited field measurements and previous ground vibration studies
- use this information to develop vibration assessment guidelines that would allow identification of the need or otherwise for a vibration management plan prior to construction starting.

The research was confined to activities associated with general mechanised road construction so blasting has been excluded.

Whenever preconstruction assessments have been carried out, the probable level of ground vibration generated by road construction equipment in many cases has been estimated using data published in *Transport and Road Research Laboratory* (*TRRL) supplementary report 328* (Martin 1977). Typical vibration characteristics of construction equipment were measured by TRRL for equipment with weights up to 75 tonnes and included representative tracked and rubber-tyred earthmoving equipment and compactors. This base-line data is, in turn, combined with the US Federal Highway Administration's (FHWA) attenuation model (Rudder 1978) to calculate the expected level of ground vibrations at various distances from the kerb edge.

The main concerns with the procedure described above relate to:

- the appropriateness of the TRRL-derived baseline vibration levels for construction activities, which apply to equipment and techniques in common use in the UK in the 1960s and 1970s, to represent existing New Zealand road construction activities. There have been significant changes to the size and design of construction equipment during this time
- a lack of controlled tests to validate the FHWA attenuation model for New Zealand conditions
- an absence of information regarding the attenuation characteristics of common New Zealand soil types.

To put the last concern in perspective, a default attenuation factor of 0.02 is used when applying the FHWA attenuation model as this value is at the lower end of what can be expected for New Zealand soil types and pertains to dense sand and gravel. Therefore, there is a degree of conservatism in the theoretically derived vibration attenuation curves that has yet to be quantified.

The above three concerns were addressed by the research, which based the methodology on the TRRL study, but under New Zealand conditions and with modern construction equipment, so that base-line vibration levels and attenuation with distance for representative soil types could be made available for the following construction equipment:

- tracked equipment, such as dozers, tractor shovels and excavators
- rubber-tyred equipment, such as motor scrapers, off-highway dump trucks and tipper lorries
- impacting equipment, such as compactors, vibratory rollers and piling rigs.

There were also enhancements. First, particular attention was paid to vibration levels generated during start up, shut down and idle operation of equipment so that any potential frequency dependent effects could be identified. Second, the measured soil attenuation characteristics were compared to common soil tests routinely performed during site investigations, such as penetration resistance using the scala penetrometer, to establish the degree of correlation. If the resulting regression equations allowed reliable estimation of soil attenuation, the need to apply conservative attenuation values for soil types that had guideline attenuation values spanning a large range would be removed.

The original aim was to measure vibration levels for a range of equipment types and associated attenuation coefficients at four sites and to measure attenuation coefficients at a further six sites. Most of the measurements were made at the sites of current road construction projects. This had the advantage of the construction activity not having to be staged and so was representative of situations likely to be encountered on future projects. However, it also had the disadvantage of having to fit in with the work being done at the time the sites were visited.

Although the site visits were timed to coincide with a period of full activity, the equipment types available were limited. Because of this, when sites were visited for attenuation measurements, the opportunity was taken to measure vibration levels of any equipment that was operating at the time. Therefore, vibration levels of construction equipment operating on the 10 sites were included, negating the need to differentiate sites where both attenuation and construction equipment vibration level measurements were made from sites where only attenuation measurements were made.

A downside to this approach was that often more than one piece of equipment was operating in the area where the vibration measurements were being carried out resulting in a significant background vibration level. In such cases, it proved difficult to isolate the vibrations attributable to a particular piece of equipment.

The report has been organised as follows. Chapter 2 considers fundamental theory of ground-borne vibrations as well as expected vibration levels for different types of construction equipment derived from a review of international literature. Chapter 3 provides a summary of the effects of ground-borne vibrations on people and structures. A description and associated findings of the measurement programme undertaken to obtain vibration levels of various construction machinery used in New Zealand road construction are given in chapter 4. Pile driving operations are specifically considered in chapter 5. Measured soil attenuation factors and their correlation to scala penetrometer readings are presented in chapter 6. Chapter 7 outlines the use of FWD output to generate site-specific vibration level predictor curves. The suggested methodologies for estimating the level of ground-borne vibrations generated at a construction site is detailed in chapter 8 along with key factors to be considered in preparing vibration mitigation plans. The principle research findings and associated recommendations are given in chapter 9. The key references are listed in chapter 10 and photographs of all the construction machinery for which vibration levels were obtained are contained in appendix A. A glossary of key terms is provided in appendix B.

Issues considered to be important and worth noting have been highlighted in ***bold italic font***.

2 Literature review

There is a considerable body of international research related to ground-borne vibrations generated from road construction activity. For example, Hiller and Crabb (2000) and Crabb and Hiller (2002) measured vibrations from several types of construction equipment in a controlled experiment and at construction sites in the UK. Jackson et al (2007), Hanson et al (2006), Jones & Stokes Associates (2004) and Hendriks (2002) provide state and federal approaches to assessing vibrations from construction equipment in the USA.

The main findings are summarised below under the sub-sections of 'fundamental theory' and 'sources' of ground-borne vibrations to facilitate ready reference.

2.1 Fundamental theory of ground-borne vibrations

2.1.1 Motion of soil particles

Dynamic excitation of soil by construction equipment causes individual soil particles to move in simple harmonic motion represented by a wave travelling through the soil. Jones & Stokes Associates (2004) model the soil system as a simple lumped parameter elastic system to describe and evaluate the response of soil to vibratory motion. This model comprises:

- a mass (representing the weight of the soil)
- a spring (representing the elasticity of the soil)
- a damper (representing material damping due to friction between soil particles and air voids).

Equation 2.1, which excludes damping, can be used to describe the vibratory motion of a mass in this simple system:

$$D(t) = D_m sin(2\pi ft) \quad \text{(Equation 2.1)}$$

where: $D(t)$ is the time varying displacement of a soil particle from rest position

D_m is the peak displacement of the mass (zero to peak)

π is ~3.1416

f is frequency of vibration (Hz)

t is time (s).

The velocity (V_m) of the mass can be determined by taking the time derivative of the displacement:

$$V_m = 2\pi f\, D_m \quad \text{(Equation 2.2)}$$

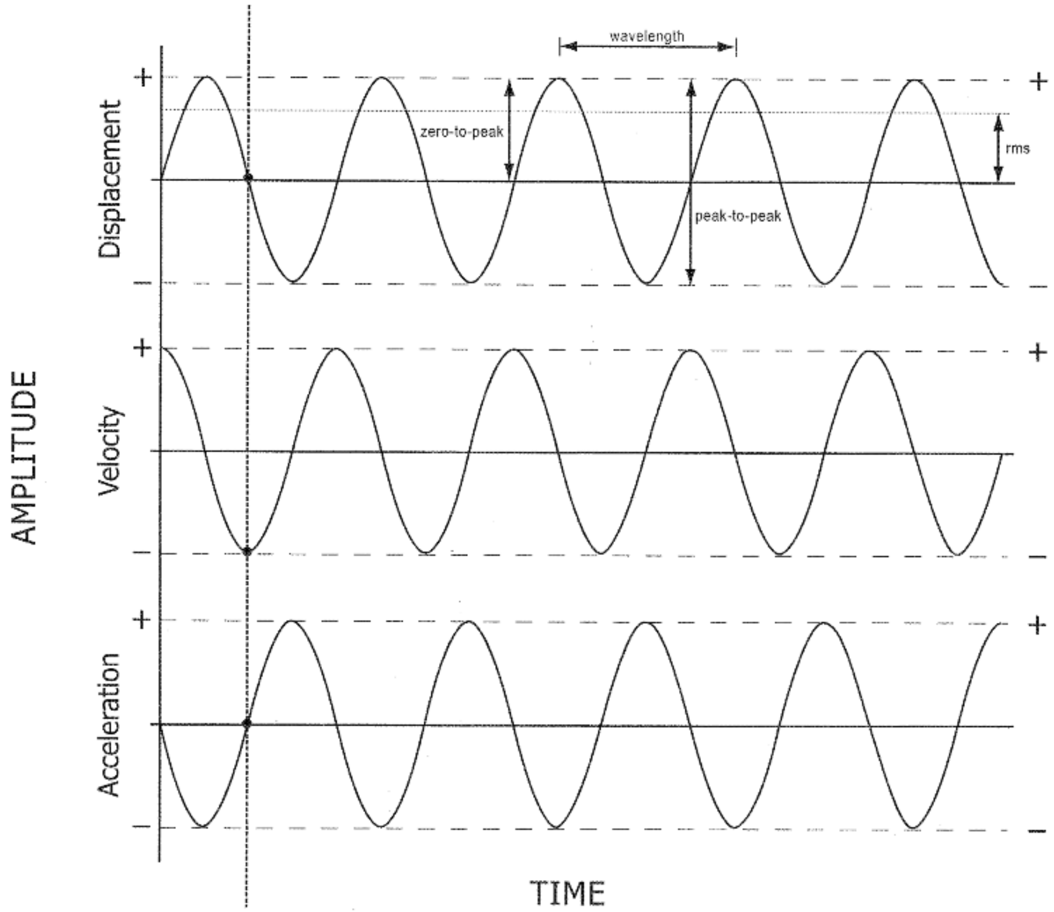
The acceleration (A_m) of the mass can be determined by taking the second time derivative of displacement or the time derivative of velocity:

$$A_m = 2\pi f\, V_m = (2\pi f)^2\, D_m \quad \text{(Equation 2.3)}$$

The maximum amplitude of motion of the soil particle can be described by peak displacement, peak velocity or peak acceleration. Any of these maximum particle motion descriptors may be found from any other as long as the frequency is known and the waveform is approximately sinusoidal.

Figure 2.1 graphically shows their interrelationship along with the quantities used to describe vibratory motion.

Figure 2.1 Quantities used to describe vibratory motion (Jones & Stokes Associates 2004)



The passage of construction vibrations forces the ground to move in an elliptical manner in three dimensions. To define this motion, three mutually perpendicular components are measured (longitudinal (radial), transverse and vertical). Typically the magnitude of the peak amplitude and the time it occurs varies between the three components and results from the presence of the various wave types discussed below.

The peak motion should only be reported as either peak component or the peak true vector sum. For construction vibration, peak particle velocity (PPV), defined as the maximum velocity component, is used as a descriptor of the effects of the wave. Therefore, PPV is a descriptor of the wave amplitude, not the wave propagation velocity.

Jackson et al (2007) note there may be times when the true vector sum of velocity components is larger in magnitude than the PPV, but this usually occurs at the same time as the largest component peak. As a consequence, PPV is the descriptor of choice for ground-borne vibrations induced by construction activity. This preference results from the close association of construction vibration with blast vibration monitoring, where particle velocity correlates with the appearance of cracking (Dowding 2000).

2.1.2 Propagation of ground-borne vibrations

When a vibratory excitation source, such as a pile driver or vibratory roller, impacts the ground, energy is transferred from the construction equipment to the ground. An intricate wave pattern is created as energy reflects and refracts off the ground surface and subsurface interfaces between dissimilar materials (Dowding 2000). This forces the soil particles to move in a complex three-dimensional manner as the energy passes.

The principal wave types that transmit vibratory energy away from a source on or near the ground to a distant receiver are as follows:

- Rayleigh (R-) waves
- shear (S-) waves
- compression (P-) waves.

Properties of these three wave types are summarised in table 2.1 below. Essentially, these three main wave types can be divided into two varieties: body waves, which propagate through the body of the soil; and surface waves, which are transmitted along a surface (usually the upper ground surface).

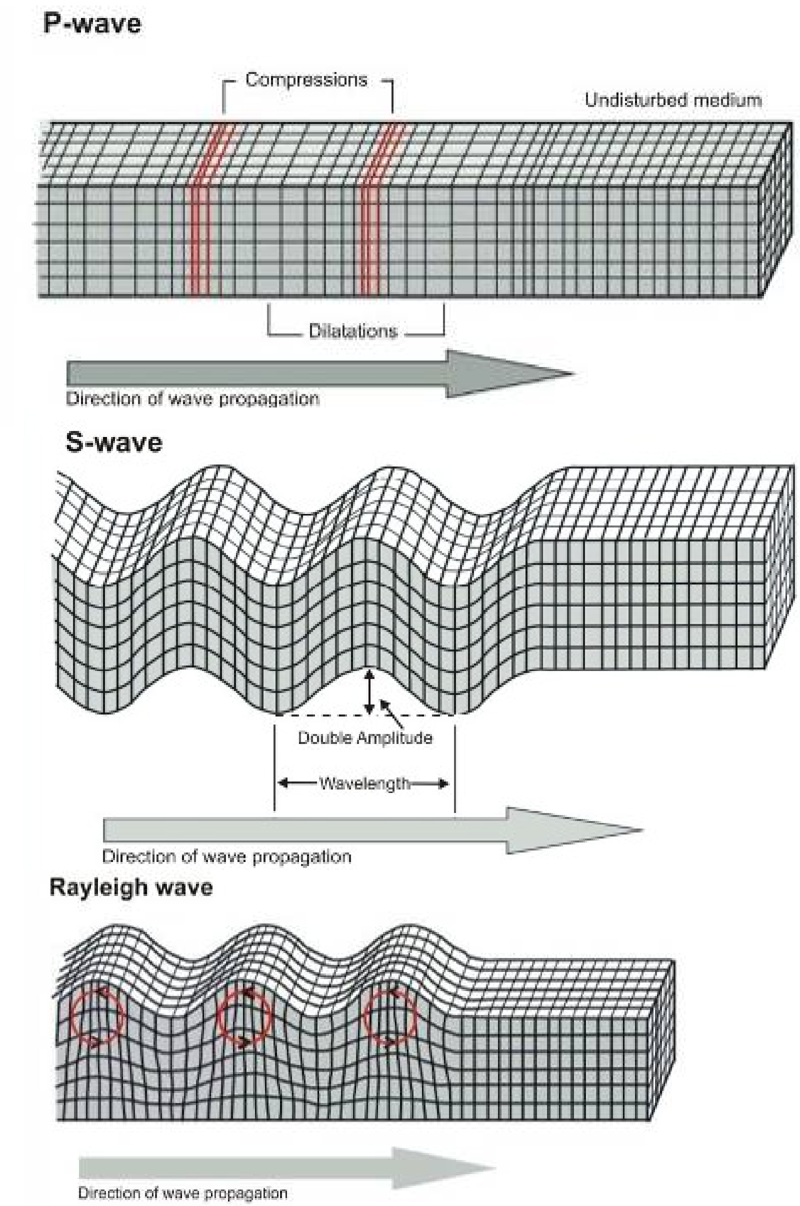
The three wave types produce radically different patterns of motion in soil and rock particles as they pass. Therefore, structures will be deformed differently by each type of wave. Figure 2.2 shows schematically the variation in particle motion with wave type.

Table 2.1 Properties of vibration wave types (from Jackson et al 2007)

Wave type	Other name for wave type	Transmission through the ground	Wave front	Particle motion	Geometric attenuation coefficient γ* (m^{-1})
Rayleigh	R-wave	Confined to surface	Cylindrical	Particles move in a circular motion in the vertical plane resulting in motion both along and perpendicular to the direction of wave propagation.	0.5
Shear	Secondary or S-wave	Body	Spherical	Particles move in the direction of the wave propagation and also perpendicular (either horizontal or vertical), causing a twisting deformation in small areas.	2
Compression	Primary or P-wave	Body	Spherical	Particles move in the direction of the wave propagation, small areas contracting and expanding in response.	2

* Applies to a measurement point on the surface

Figure 2.2 Deformation characteristics of P-, S- and R-waves (SOS-LIFE Earthquake Early Warning System 2004–2005: www.lamit.ro/earthquake-early-warning-system.htm)



Construction-related ground impacts produce predominately body waves. These body waves propagate outward in a spherical manner until they intersect at a boundary, such as another layer (rock or soil) or ground surface. At this intersection, S- and R- waves are produced.

At small distances from the vibration source, all three wave types will arrive together and greatly complicate wave identification; whereas at large distances, the more slowly moving S- and R-waves begin to separate from the P-wave and allow identification.

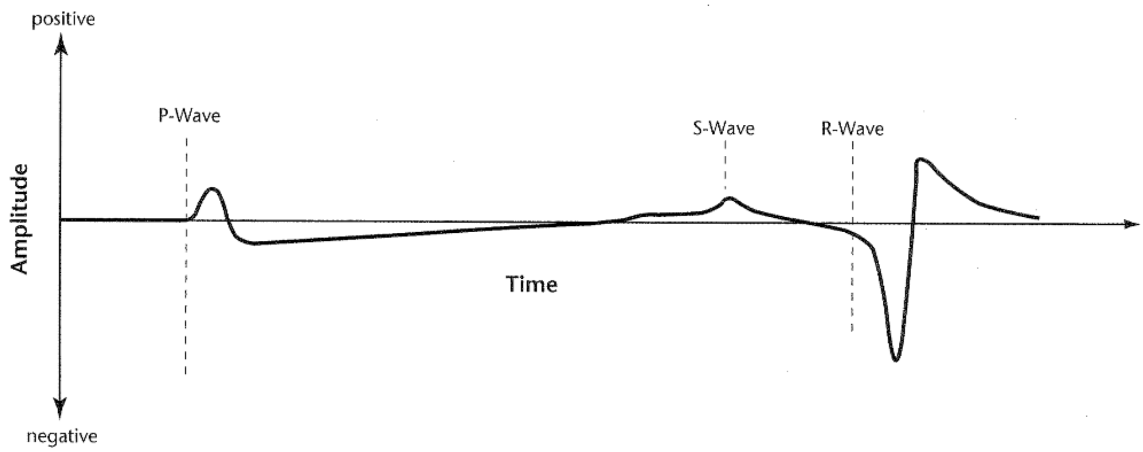
The P-, S- and R- waves travel at different speeds. The P-wave is the fastest, followed by the S-wave, then the R-wave.

Along the surface of the ground, the P- and S- waves decay more rapidly than the R-wave. Therefore, the R-wave is the most significant disturbance along the surface of the ground and may be the only clearly distinguishable wave at large distances from the source. For example, R-waves account for 67% of the total energy, S- waves for 26% and P-waves for 7% when the exciting force is applied vertically to the propagation direction (Richart et al 1970).

At higher frequencies, the R-wave, however, may not be identifiable because inhomogeneities and layering in the ground complicate the propagation of these waves.

The characteristic wave system for a short-duration ground disturbance, when the vibration source and the receiving point are a few metres to a few hundred metres apart, is shown in figure 2.3. It can be seen that the P-wave arrives first at the receiver, followed by the S- wave and finally the R- wave.

Figure 2.3 Characteristic wave system from a surface point source (Jones & Stokes Associates 2004)



Energy from surface waves is conserved with distance relative to energy from either type of body wave (Jackson et al 2007) as shown in figure 2.3 by the surface wave (R-wave) having a larger amplitude than the two body waves (P-wave and S-wave). This is due to the difference in geometry between surface waves and body waves. R-waves are surface waves that travel in a cylindrical wavefront. This means that most of an R-wave's energy is confined to a volume below the surface that is one wavelength deep. However, P- and S-waves are body waves. Body waves radiate in all directions into the ground in a hemispherical wavefront. This hemispherical wavefront has a much larger surface area than the cylindrical wavefront of a surface wave. Therefore, conservation of energy dictates that R-waves transmit more energy between two points on the surface than either of the body wave types. This difference in geometric attenuation between surface and body waves translates to the lower geometric attenuation coefficient (0.5 cf 2) listed in table 2.1.

2.1.3 Propagation velocity

Propagation velocity is an important factor because it is an indirect measure of rock/soil properties that affect the decay of PPVs as well as wavelengths. Generally, propagation velocity increases with increasing soil stiffness. A wave propagates more quickly through a hard, dense material than through a soft, pliable material. Jointing and weathering of rock masses greatly affect propagation velocities through changing rock stiffness. Therefore, propagation velocity generally increases with increasing depth because the intensity of jointing decreases.

Propagation through rock and soil provides an even greater propagation velocity contrast than wave type as shown by the comparison given in table 2.2.

Table 2.2 Estimated propagation velocities for different materials over heavily jointed to non-jointed condition (Dowding 2000)

Material	Wave velocity (m/s)	
	Compression (P-) wave	Shear (S-) wave
Limestone	2000–5900	1000–3100
Metamorphic rocks	2100–3500	1000–1700
Basalt	2300–4500	1100–2200
Granite	2400–5000	1200–2500
Sand	500–2000	250–850
Clay	400-1700	200-800

Lower value in range pertains to heavily jointed condition
Upper value in range pertains to non-jointed condition

As the vibration of any waveform can be expressed as overlapping sinusoidal waves, propagation velocity (c) is related to frequency (*f*) and wavelength (λ) as follows:

$$\frac{c}{f} = \lambda \qquad \text{(Equation 2.4)}$$

Propagation velocity should not be confused with particle velocity. Dowding (2000) uses the analogy of the motion of a bobbing cork during a passing wave to distinguish these two velocities. The propagation velocity is the speed with which the wave passes the cork, while the particle velocity is the speed with which the cork moves up and down.

2.1.4 Wave attenuation

Attenuation describes the decay of vibrations with distance from source. The focus will be on R-waves as ground-borne vibrations from construction equipment propagate predominately by this wave type.

2.1.4.1 Decay with depth

R-waves decay exponentially with depth. The depth of penetration of the wave is approximately equal to its wavelength (λ), which from equation 2.2 is a function of the wave velocity of the material and frequency of the wave. The decay in wave amplitude with depth is given by the following relationship:

$$A(z) = A_0 e(\frac{-z}{z_0}) \qquad \text{(Equation 2.5)}$$

where: $A(z)$ = R-wave amplitude at depth z

A_0 = amplitude at the surface

z_0 = characteristic penetration depth, which is proportional to wavelength

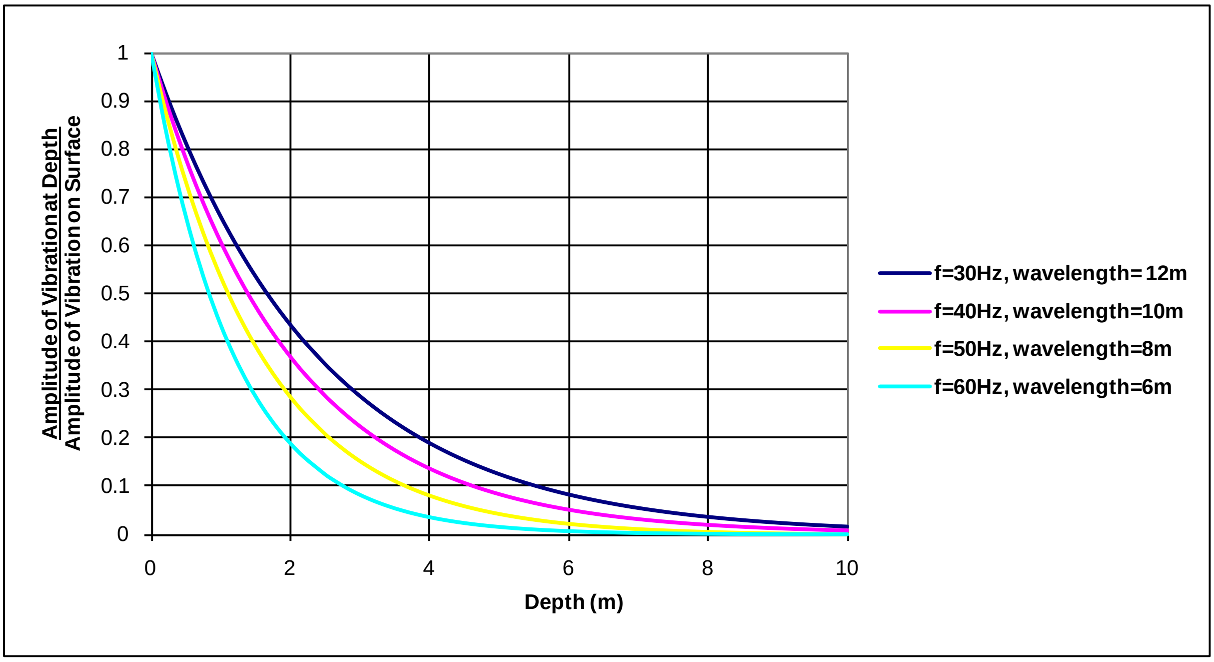
Figure 2.4 shows the decay of R-waves with depth and frequency for a wave velocity of 380m/s (a value typical of a dense, unsaturated sand). As can be seen, the decay of R-waves with depth is more rapid as the frequency of the wave increases.

The depth of penetration of the wave is approximately equal to its wavelength (Jackson et al 2007). Figure 2.4 shows that at one sixth of a wavelength below the surface, the amplitude of vibration reduces to half the amplitude at the surface. Figure 2.4 also shows that 99% of an R-wave's energy travels in the top one wavelength of the soil.

The wavelength, and therefore the depth of penetration is a function of the:

- wave velocity of the material
- frequency of the construction equipment that causes the vibration.

Figure 2.4 Attenuation of R-waves with depth for dense, unsaturated sand (Jackson et al 2007)



2.1.4.2 Decay with distance

The amplitude of R-waves decreases with distance from source. This attenuation or decay is produced by two mechanisms:

- geometrical spreading
- material damping.

Geometrical spreading is independent of material properties. It refers to the increasing area (for body waves) or length (for surface waves) over which the wavefront spreads as the vibration propagates away from the source (Hiller and Hope 1998). Equation 2.6 describes attenuation by geometrical spreading.

$$V_2 = V_1 \left(\frac{R_1}{R_2}\right)^{\gamma} \qquad \text{(Equation 2.6)}$$

where: γ = 0.5 for R-waves or γ = 2 for P- or S- waves (refer table 2.1)

V_1 = the particle velocity at distance R_1

V_2 = the particle velocity at distance R_2

In a perfectly elastic medium, equation 2.7 would exactly describe the attenuation of R-waves.

$$V_2 = V_1\sqrt{\frac{R_1}{R_2}} \qquad \text{(Equation 2.7)}$$

In reality, vibrations attenuate more than equation 2.7 predicts. For each cycle of motion or wavelength, λ, travelled, the wave loses a small amount of energy that is required to overcome friction. This hysteretic loss of energy during one cycle of deformation is called material damping since it is a function of the material's deformational properties (Dowding 2000). Material damping has been shown to be proportional to the logarithm of the distance travelled as in equation 2.8.

$$V_2 = V_1 e^{-\alpha(R_2-R_1)} \qquad \text{(Equation 2.8)}$$

where: α = attenuation coefficient (m^{-1}), which increases with dominant frequency

Combined geometric and material damping of waves can be described by equation 2.9

$$V_2 = V_1\left(\frac{R_1}{R_2}\right)^{\gamma} e^{-\alpha(R_2-R_1)} \qquad \text{(Equation 2.9)}$$

Hiller and Crabb (2000) adapted equation 2.9 for a line source as shown in equation 2.10. A point source is a good approximation for a roller drum when at a distance from the roller. Close to the roller, a line source is more accurate since energy is transmitted into the ground along the line of contact between the fill and vibrating drum. This line of contact is the length of the roller drum, *w*, (Hiller and Crabb 2000). In practice, equation 2.10 does not produce significantly different predictions to equation 2.9 and is more complicated to use.

$$V_2 = V_1\left(\frac{R_1 + w}{R_2 + w}\right)^{\gamma} e^{-\alpha(R_2-R_1)} \qquad \text{(Equation 2.10)}$$

2.1.4.3 Determination of soil attenuation coefficient

Amick (1999) describes two approaches to fit equation 2.9 to measured data:

1. Neglect material damping (assume $\alpha = 0$) and find a geometric attenuation coefficient (γ) that makes equation 2.6 fit the measured data. This approach models energy transfer away from the source as a combination of surface waves and body waves with no loss due to material damping. Different values of the geometric attenuation coefficient for different soil types represent transport of energy away from the source by different combinations of surface and body waves.
2. Assume R-waves dominate (ie assume $\gamma = 0.5$) and find a material damping coefficient (α) that makes equation 2.9 fit the measured data. This approach models energy transfer away from the source as a purely surface wave with losses due to material damping.

Approach 2 is preferred because it generally provides a better fit (Hiller and Crabb 2000).

A summary of published geometric attenuation coefficients (γ) using the first of these approaches is given in table 2.3.

Table 2.3 Summary of published geometric attenuation coefficients, γ, assuming no material attenuation (Amick 1999)

Investigator	Soil type	Geometric attenuation coefficient, γ
Wiss	Sands	1.0
	Clays	1.5
Brenner and Chittikuladilok	Surface sands	1.5
	Sand fill over soft clays	0.8 – 1.0
Attewell and Farmer	Various soils, generally firm	1.0
Nicholls, Johnson and Duvall	Firm soils and rock	1.4 – 1.7
Martin	Clay	1.4
	Silt	0.8
Amick and Ungar	Clay	1.5

Dowding (2000) provides typical values for the coefficient of attenuation for material damping (α) as tabulated in table 2.4. These values were found by fitting equation 2.9 to measured vibrations as per approach 2.

Table 2.4 Typical values of attenuation coefficient, α (Dowding 2000)

Description of material	Attenuation coefficient α (m^{-1})	
	5Hz	50Hz
Weak or soft soils (shovel penetrates easily) peat, loose dune sand and topsoil	0.01 – 0.03	0.1 – 0.3
Competent soils (can dig with shovel) sands, clay, silt and completely weathered rock.	0.003 – 0.01	0.03 – 0.1
Hard soil (need pick to dig) sandstone, consolidated clay and moderately weathered rock	0.0003 – 0.003	0.003 – 0.03
Hard rock (difficult to break with hammer) unweathered rock	< 0.0003	< 0.003

A summary of measured values of α for various soil types prepared by Jones & Stokes Associates (2004) is given in table 2.5. The attenuation coefficients in table 2.5 are generally in agreement with those in table 2.4 and are appropriate for use with equation 2.9.

Table 2.5 Summary of attenuation coefficients, α, for various soil types (Jones & Stokes Associates 2004)

Investigator	Soil type	Attenuation coefficient α (m^{-1})
Forssblad	Silty gravelly sand	0.13
Richart	100–150mm concrete slab over compacted granular fill	0.02
Woods	Silty fine sand	0.26
Barkan	Saturated fine grain sand	0.010
	Saturated fine grain sand in frozen state	0.06
	Saturated sand with laminae of peat and organic silt	0.04
	Clayey sand, clay with some sand, and silt above water level	0.04
	Marly chalk	0.1
	Loess and loessial soil	0.1
	Saturated clay with sand and silt	0.0 – 0.12
Dalmatov et al	Sand and silts	0.026 – 0.36
Clough and Chameau	Sand fill over bay mud	0.05 – 0.2
	Dune sand	0.026 – 0.065
Peng	Soft Bangkok clay	0.026 – 0.44
Hendriks	Sand-silt, clayey silt, silty sand	0.021

Another summary of measured soil attenuation coefficients given in Rudder (1978) has been reproduced as table 2.6.

Table 2.6 Values of attenuation coefficient, α, for different soil types (Rudder 1978)

Soil type	Attenuation coefficient α (m^{-1})
Moist clay	0.025 – 0.25
Clayey soil	0.025 – 0.25
Silty clay	0.019 – 0.43
Wet clay	0.31 – 0.5
Loess at natural moisture	0.04 – 0.13
Dry sand	0.007 – 0.070
Dense sand & gravel	0.015 – 0.045
Gravel + sand & silt	0.023 – 0.053
Fine grained sand, water saturated	0.09 –0.30
Fine grained sand, water saturated & frozen	0.05 – 0.17

The values of α given in tables 2.4, 2.5 and 2.6 show good conformity. The other point to note is that the range of values for some soil types can be very large, covering an order of magnitude. This highlights that soil attenuation coefficients are highly site specific due to the complex nature of soil characteristics and so should be measured when used to assess expected vibration levels from construction activity (Hunaidi 2000). Any vibration assessment guideline should therefore address this issue, which is considered further in chapter 6.

Figures 2.5 and 2.6 graphically compare output from the two attenuation models represented by equations 2.6 and 2.9 for a range of attenuation coefficients.

Figure 2.5 Influence of α and γ on vibration amplitude V_2 with distance relative to amplitude V_1 measured 2m from the source

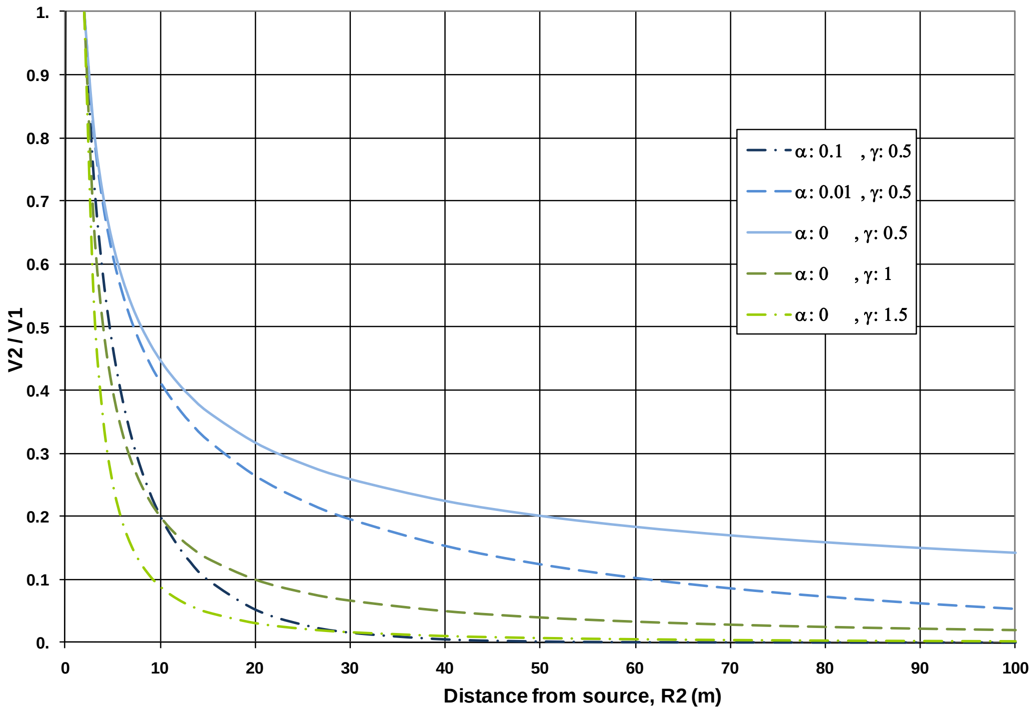
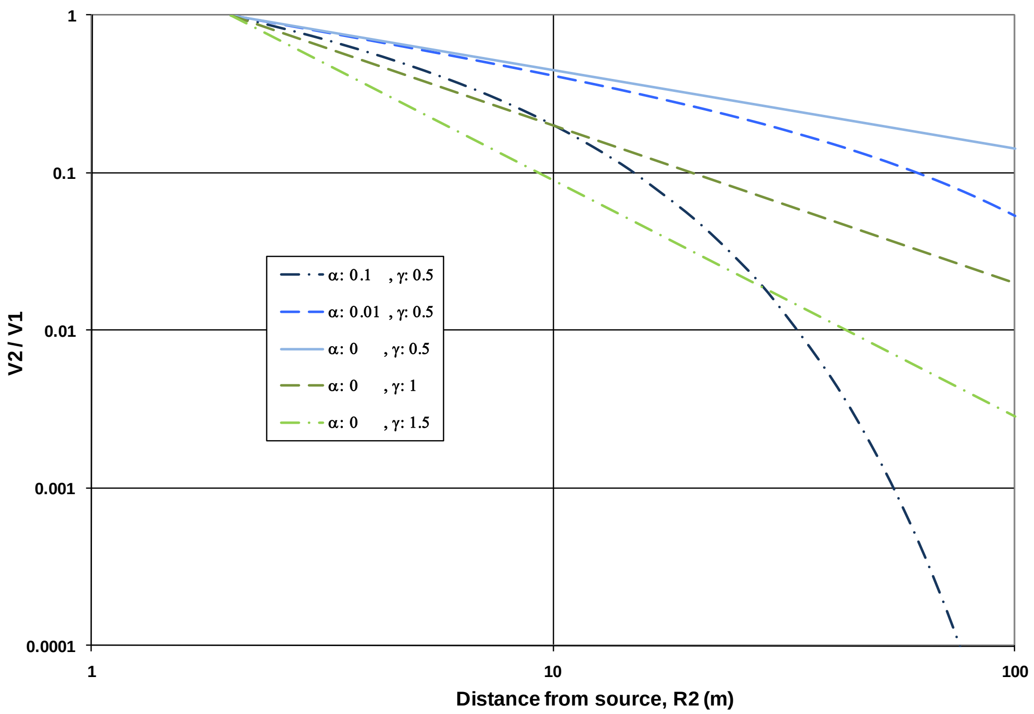


Figure 2.6 Data in figure 2.6 represented as a log-log plot



The green lines in figures 2.5 and 2.6 show approach 1 for a range of geometric damping coefficients that represent energy transfer modelled by a varying mixture of surface waves and body waves. A value of γ closer to 2 represents a higher proportion of energy carried by body waves whereas a value of γ closer to 0.5 represents a lower proportion of energy carried by body waves.

The blue lines in figures 2.5 and 2.6 show approach 2 for a range of material damping coefficients that represent the different absorption losses of the vibratory energy carried by R-waves through three different soil types ranging from theoretical lossless soil (α = 0m^{-1}) to silty, gravelly sand or loessial soil (α = 0.1m^{-1}).

Figure 2.6 shows that as geometric spreading due to body waves dominates energy propagation, ie γ close to 2, the vibration far from the source is less than that modelled with purely R-waves even if some material damping is included (α = 0.01m^{-1}). This is due to the different propagation mechanism, which carries energy away from the source into lower layers of the elastic hemispherical space. However with strong material damping included (α = 0.1m^{-1}), attenuation is increasingly dominated by material damping with distance, especially beyond 30m.

2.1.4.4 Influence of frequency on material damping

Amick (1999) notes that vibrations of a higher frequency decay faster than those of a lower frequency and so a frequency dependent soil propagation model has been proposed in which the material damping coefficient α is a function of:

- frequency, f, and
- a frequency independent material property of the soil, ρ.

Amick (1999) uses ρ to describe attenuation as in equation 2.11.

$$\alpha = \frac{\eta}{c}\,\pi\, f = \rho\pi f \qquad \text{(Equation 2.11)}$$

Amick (1999) defines $\rho = \eta/c$ as a frequency independent material property of the soil and uses ρ to classify soil types without requiring the measurement or estimation of the loss factor η or the wave propagation velocity c. Therefore, a complicated two parameter curve fitting exercise reduces to a simple one parameter fit.

Equation 2.11 when combined with equation 2.9 results in equation 2.12. Amick (1999) presents ρ for a range of soil types as shown in table 2.7.

$$V_2 = V_1\left(\frac{R_1}{R_2}\right)^{\gamma} e^{-\rho\pi f(R_2-R_1)} \qquad \text{(Equation 2.12)}$$

Hiller and Crabb (2000) propose a similar frequency dependent coefficient of attenuation for vibrations through London clay and as-dug hoggin (similar to pit-run gravel, aggregate/clay mix) as described by equation 2.13.

$$\alpha = 0.01 ln\,(f) \qquad \text{(Equation 2.13)}$$

However, over the range of frequencies commonly caused by construction equipment, the frequency dependence is usually considered weak. Therefore, α is often assumed independent of frequency (Jackson et al 2007). This does not mean frequency should be ignored altogether, merely that it is of less concern than distance from source when assessing construction vibrations. For sensitive sites, Hiller and Crabb (2000) consider it prudent to take cognisance of the operating frequency of the equipment.

Table 2.7 Summary of frequency dependent attenuation coefficient (α) and frequency independent soil property (ρ) for various soil types (adapted from Amick 1999)

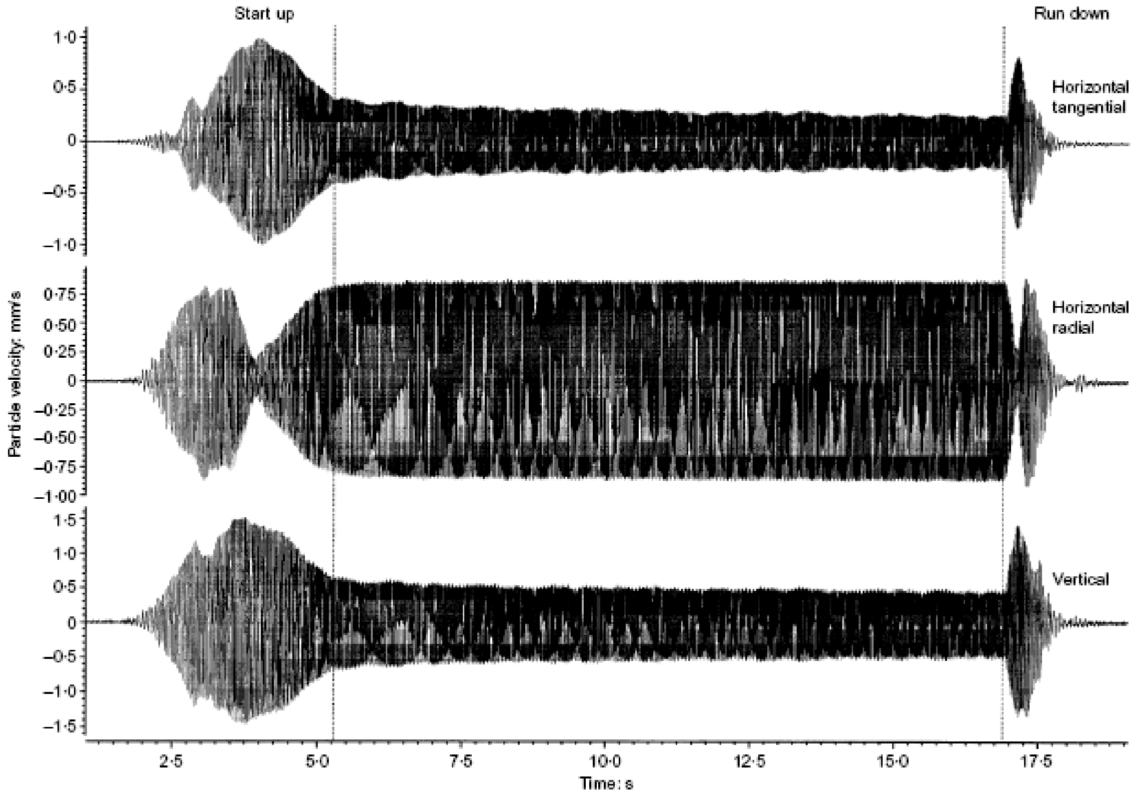
Class	Description of soil	Attenuation coefficient, α, at 5 Hz (m^{-1})	Frequency independent soil property, ρ (s/m)
I	Weak or soft soils (soil penetrated easily: loess soils, dry or partially saturated peat and muck, mud, loose beach sand and dune sand, recently ploughed ground, soft spongy forest or jungle floor, organic soils, topsoil	0.01 – 0.03	$6x10^{-4}$ - $2x10^{-3}$
II	Competent soils (can dig with shovel): most sands, sandy clays, silty clays, gravel, silts, weathered rock	0.0003 – 0.01	$2x10^{-5}$ – $6x10^{-4}$
III	Hard soils (cannot dig with shovel, must use pick to break up): dense compacted sand, dry consolidated clay, consolidated glacial till, some exposed rock	0.0003 – 0.003	$2x10^{-5}$ – $2x10^{-4}$
IV	Hard, competent rock (difficult to break with a hammer): bedrock, freshly exposed hard rock	< 0.0003	< $2x10^{-5}$

2.1.5 Start up and shut down phase of construction equipment

As powered construction equipment starts up or shuts down, the vibrations generated change frequency (Hiller and Crabb 2000). As a result, the vibration levels at start up and shut down may be considerably higher than under normal running. This is due to the frequency of operation being lower during the start-up and shut down phases, which translates into the soil attenuation being lower.

This phenomenon is shown for a vibrating roller in figure 2.7.

Figure 2.7 Measured vibration from a vibrating roller, showing start up, steady state operation and run down (Crabb and Hiller 2002)



As can be seen in figure 2.7, the start up and run down phase can cause vibrations to be up to twice as strong as during regular, steady state, operation. Therefore, in sensitive areas, there is a need to carefully plan where construction equipment should be left parked.

2.1.6 Empirical equations for predicting vibrations from blasting, pile driving and vibratory compaction

Several empirical equations have been proposed that describe vibration attenuation through soil for specific cases.

The scaled distance approach, presented as equation 2.14, is commonly used for blasting and pile driving operations (Svinkin 1999 and Wiss 1981).

$$V = k\left(\frac{R}{\sqrt{W}}\right)^{-n} \qquad \text{(Equation 2.14)}$$

where: V = PPV at distance R from the source

W = the energy of source or rated energy of impact hammer

k and n = parameters found by plotting V versus R on a log-log plot

Wiss (1981) notes that *k* and *n* are unique for each source and soil type combination and this approach cannot be used without detailed site-specific measurements.

Several versions of this approach have been proposed by, for example, New (1986), Svinkin (1999) and Hanson et al (2006) for various types of pile driving and blasting. Generally, these approaches are inappropriate for assessing vibrations from other construction equipment as they require knowledge of the energy level at the source of the vibration.

Hiller and Crabb (2000) measured vibration levels from vibratory rollers at several sites in the UK and derived the empirical equation given by equation 2.15. The soils at these sites were representative of the soil types found in the UK. Specifically, sandstone, glacial till, tuff, boulders on limestone, mudstone and sandy clay on sand and gravel were included in the trial.

Equation 2.15 fits Hiller and Crabb's (2000) measured vibrations on these soils but does not take into account the theoretical geometric spreading or soil properties at the site. Therefore, equation 2.15 is only suitable for predicting vibrations from vibratory rollers on UK soil types. As a consequence, it would be inappropriate to apply equation 2.15 to the volcanic soils of the central North Island or dune sand found in coastal parts of New Zealand.

$$V = k_s \sqrt{n} \left[\frac{A}{R + w}\right]^{1.5}$$ (Equation 2.15)

where: k_s = 75 with a 50% probability of the vibration level being exceeded

k_s = 143 with a 33% probability of the vibration level being exceeded

k_s = 276 with a 5% probability of the vibration level being exceeded

n is the number of vibrating drums (ie 1 or 2)

A is the nominal amplitude of the vibrating drums

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

2.2 Vibration levels from construction activities

The review of published measurements of vibrations levels from construction activities was confined to activities associated with general mechanised road construction and so blasting was excluded.

Hiller and Crabb (2000) undertook a comprehensive review of ground vibration from compaction, piling, tunnelling and other mechanised construction and ground improvement techniques. As part of this review, an energy-related approach to predicting the level of vibration 2m from the source was derived. In the case of vibratory rollers, this approach assumes that, during each cycle, the vibrating drum rises through a distance equal to the nominal amplitude (*A*) and dropped to the ground. The potential energy gained and dissipated during this cycle approximates the energy input to the soil. The weight, which includes the weight of the drum and the contribution from all the equipment, is calculated from the static linear load (L_s) and the drum width (*w*). The peak particle velocity at 2m from the source (V_{2m}) would then be proportional to the square root of the energy term shown in equation 2.16.

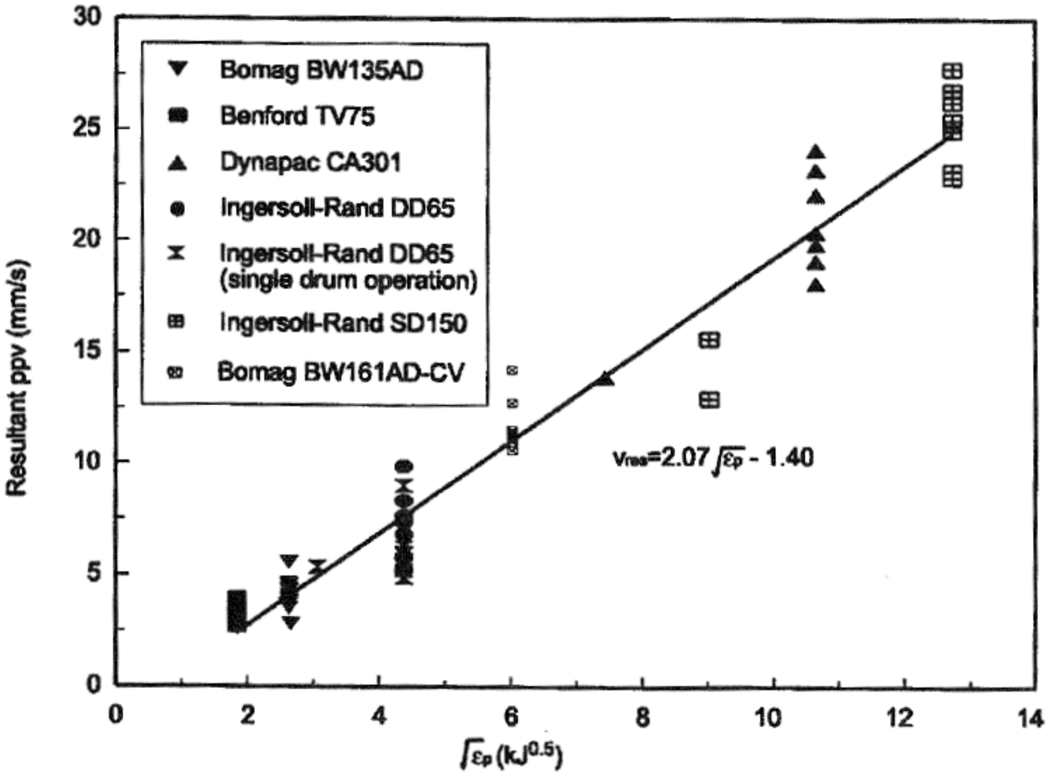
$$\varepsilon_p = AL_s wg$$ (Equation 2.16)

where: ε_p = the energy term or the energy of one cycle of the vibrating drum;

g = 9.81 m/s2, is the acceleration due to gravity

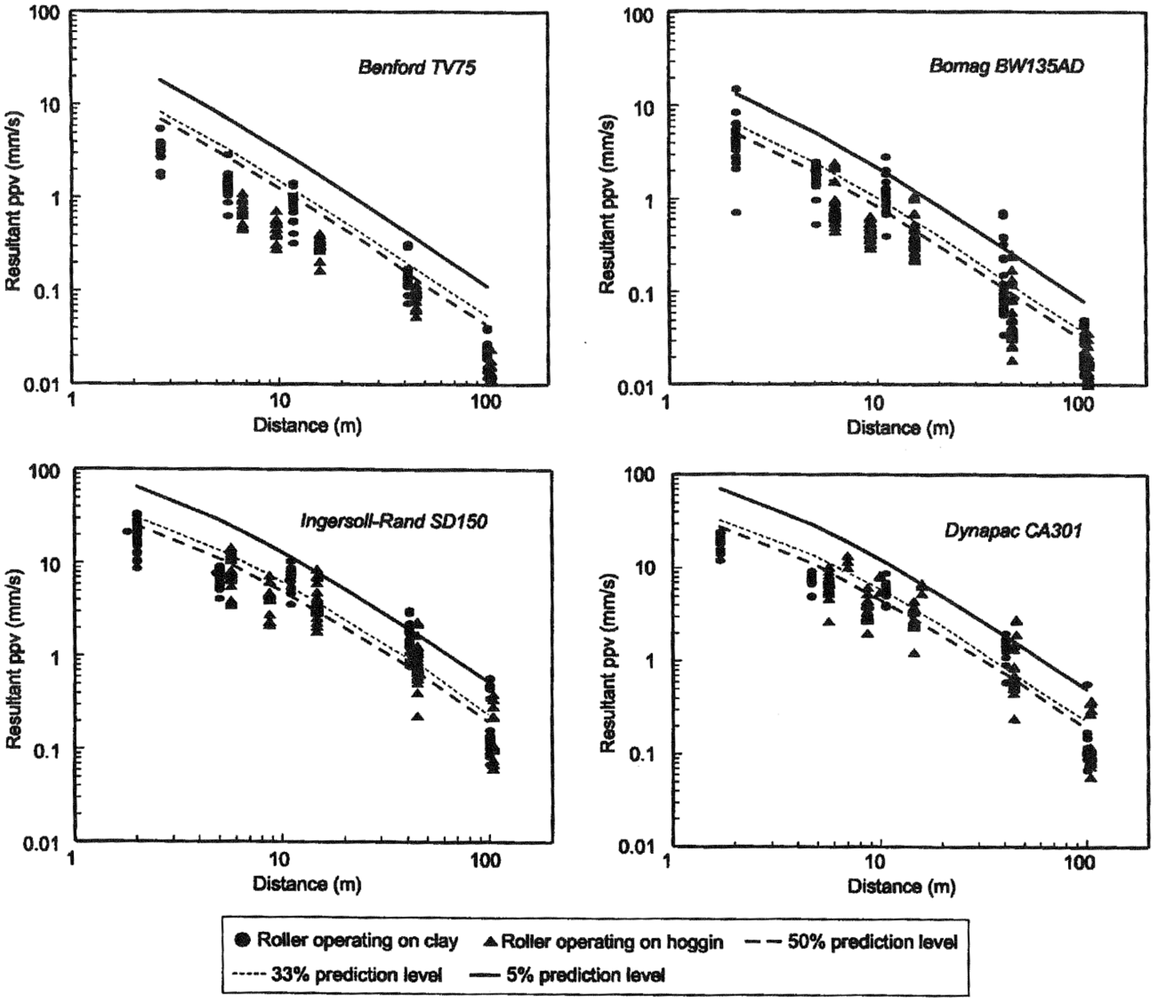
Using this theoretical energy approach, Hiller and Crabb (2000) found a linear relationship between the square root of the energy term and the PPV measured 2m from the source as shown in figure 2.8. The vibration levels plotted are from construction equipment in a controlled experiment on London clay.

Figure 2.8 Relationship between the peak particle velocity (PPV) recorded 2m from the roller and the energy term ε_p (Hiller and Crabb 2000)



Another important finding from Hiller and Crabb is that for a specific type of construction equipment, there can be a significant difference in the level of ground-borne vibration generated depending on the make and model. This is illustrated in figure 2.9 for a selection of vibratory compactors when operated on two different soil types, clay and hoggin.

Figure 2.9 Measured vibration levels from four different vibratory compactors and fitted probability of exceedance curves (Hiller and Crabb 2000)



With reference to figure 2.9, the three lines fitted to the measured vibration levels pertain to the predicted vibrations levels which would not be expected to be exceeded in 50%, 33% and 5 % of cases. Equation 2.10 was used to generate the fitted lines.

Annex E of British Standard BS 5228-2:2009 *Code of practice for noise and vibration control on construction and open sites – part 2: Vibration* contains the empirical formulae derived by Hiller and Crabb (2000) from field measurements relating resultant PPV with a number of other parameters for vibratory compaction, percussive and vibratory piling, dynamic compaction, the vibration of stone columns and tunnel boring operations. These prediction equations are based on the energy approach described above. Use of these empirical formulae enables resultant PPV to be predicted and for some activities (vibratory compaction, vibratory piling and vibrated stone columns) they can provide an indicator of the probability of these levels of PPV being exceeded.

BS 5228-2:2009 also provides case history data on vibration levels for piling and ancillary operations.

A summary of vibration levels that have been measured for various types of construction equipment under a variety of conditions has been prepared by Hanson et al (2006). Because of its comprehensive nature, this summary has been reproduced as table 2.8 for ready reference. The vibration source levels tabulated should provide a reasonable estimate for a wide range of conditions.

Table 2.8 Measured vibration source levels for construction equipment (adapted from Hanson et al 2006)

Equipment	PPV at 7.6m (mm/s)
Pile driver (impact)	38.6 (upper range)
	16.4 (typical)
Pile driver (sonic)	18.6 (upper range)
	4.3 (typical)
Clam shovel drop (slurry wall)	5.1
Hydromill (slurry wall)	0.2 (in soil)
	0.4 (in rock)
Vibratory roller	5.3
Hoe ram	2.3
Large bulldozer	2.3
Caisson drilling	2.3
Loaded trucks	1.9
Jackhammer	0.9
Small bulldozer	0.1

3 Effects of ground-borne vibrations

Building vibrations caused by mechanised construction activity are unlikely to be a health and safety concern provided the separation distance between the vibration source and the building is sufficiently long. Typically, these vibrations are more likely to cause annoyance.

Vibrations may be unacceptable to occupants of buildings because of:

- annoying physical sensations produced in the human body
- interference with activities such as sleep and conversation
- rattling of window panes and loose objects
- fear of damage to the building and its contents.

Experience has shown that people living in buildings are likely to complain even if vibration levels are only slightly above the perception threshold, the major concern being fear of damage to the building or its contents. This tolerance level varies widely from person to person and area to area (Hunaidi 2000).

Ground-borne vibration is almost never annoying to people who are outdoors. Although the motion of the ground may be perceived, without the effects associated with the shaking of a building, the motion does not provoke the same adverse human reaction (Hanson et al 2006).

Vibration levels that cause damage to buildings are an order of magnitude greater than the human perception level. Therefore, occupants of buildings would find potentially damaging vibrations to be extremely annoying because of their high level. As a consequence, the focus is on managing vibrations to a level in residential environments that is unlikely to cause complaint.

3.1.1 Types of vibration

The Department of Environment and Conservation (New South Wales) in their publication *Assessing vibration: technical guideline* (DEC 2006) classifies vibration as continuous, impulsive or intermittent as follows:

- Continuous vibration continues uninterrupted for a defined period (usually throughout day-time and/or night-time). Examples include steady road traffic or continuous construction activity such as tunnel boring.
- Impulsive vibration is a rapid build up to a peak followed by a damped decay but may or may not involve several cycles of vibration (depending on frequency and damping). It can also consist of a sudden application of several cycles at approximately the same amplitude, providing the duration is short, typically less than two seconds. It relates to infrequent activities that create up to three distinct vibration events in an assessment period. An example would be occasional loading and unloading.
- Intermittent vibration can be defined as interrupted periods of continuous (eg a drill) or repeated periods of impulsive vibration (eg a pile driver), or continuous vibration that varies significantly in magnitude. It may originate from impulsive sources (eg pile drivers) or repetitive sources (eg pavement breakers) or sources that operate intermittently, but which would produce continuous vibration if operated continuously (eg heavy traffic passing by).

  This type of vibration is best assessed on the basis of vibration dose values as defined in British Standard BS 6472-1:2008 *Guide to evaluation of human exposure to vibration in buildings, part1: Vibration sources other than blasting.*

Most vibration guidelines published by standards organisations for avoiding adverse response from building occupants involve several differing thresholds and unfortunately for impulsive event analysis still mostly rely on continuous motion test results. Dowding (2000) highlights that more work is needed to establish a scientific basis for the applicability of these standards for situations where vibration events are intermittent or are separated but may occur several times per day. For example, in the case of nominally continuous sources of vibration such as traffic, vibration is perceptible at around 0.3 – 0.5m/s PPV and above these values may become disturbing or annoying. However, higher levels of vibration are typically tolerated for single events or events of short duration occurring during daytime. For example, blasting and piling, two of the primary sources of vibration during construction, are typically tolerated at vibration levels up to 12mm/s PPV and 2.5mm/s PPV respectively (NRA 2004).

Therefore, it is important to remember that even though called 'standards', the standards discussed below are guides for the evaluation of existing or planned construction activities.

3.2 Effect of vibration on people

Human perception and response to ground vibration varies widely. It depends on individual sensitivity, the frequency, PPV, duration, and on whether or not the event is expected and if so, whether the vibration is expected to cause damage. A person working in a factory is likely to be less sensitive to vibration than that same person would be quietly reading a book in a residential home. Most routine complaints of vibration come from people who are ill, elderly, or are engaged in a vibration sensitive hobby or activity (Jackson et al 2007).

Even low-level vibrations may cause annoying secondary effects such as rattling windows and doors. While these secondary effects may be irritating they are not likely to cause property damage (Jackson et al 2007). Homeowners who are aware of this are less likely to complain about the vibration.

The vibration standard for evaluating human exposure to vibration in buildings that is traditionally applied in New Zealand and has been incorporated in a number of district plans is NZS/ISO 2631-2:1989: *Evaluation of human exposure to whole body vibration, part 2: Continuous and shock induced vibration in buildings (1 to 80Hz).* However this standard is no longer considered valid, as it was replaced in 2003 by an informative only standard containing no vibration criteria, and the Standards New Zealand's adoption of the 1989 standard was withdrawn in 2005. Therefore, Standards New Zealand has no current standard for providing guidance for evaluating human response to building vibration.

A review of relevant international vibration standards (Whitlock 2010) identified only one current standard that specifically considers human response to vibrations from construction activity. This standard is BS 5228-2:2009 and is described below along with BS 6472-1:2008, which has been recommended for evaluating intermittent vibrations (DEC 2006).

3.2.1 British Standard BS 5228-2:2009

'BS 5228-2:2009 is a comprehensive document covering many aspects of prediction, measurement, assessment and control of vibration from construction works' (Whitlock 2010).

Annex B of the standard provides valuable information on people's expectations and response to construction vibration. This is reproduced as table 3.1.

Table 3.1 Guidance on effect of vibration levels from BS 5228-2:2009

Vibration level (PPV, mm/s)	Effect
0.14mm/s	Vibration might be just perceptible in the most sensitive situations for most vibration frequencies associated with construction. At lower frequencies, people are less sensitive to vibration.
0.3mm/s	Vibration might be just perceptible in residential environments.
1.0mm/s	It is likely that vibration of this level in residential environments will cause complaint, but can be tolerated if prior warning and explanation has been given to residents.
10mm/s	Vibration is likely to be intolerable for any more than a very brief exposure to this level.

These guidance vibration levels are generally consistent with the recommendations of NZS/ISO 2631-2:1989 and are in terms of PPV, which is the vibration parameter routinely measured when assessing potential building damage. Furthermore, since many of the empirical equations for predicting construction-related vibrations give estimates in terms of PPV, the consequences of any predicted levels in terms of human perception and disturbance can be readily understood through direct comparison with the BS 5228-2:2009 guidance vibration levels.

3.2.2 British Standard BS 6472-1:2008

'BS 6472-1:2008 is not widely adopted in New Zealand, but is attractive for use in assessment of intermittent vibration effects due to its dose-response metric, vibration dose value (VDV)' (Whitlock 2010). This metric is defined in equation 3.1.

$$VDV = \left(\int_0^T a^4\,(t)dt\right)^{0.25} \qquad \text{(Equation 3.1)}$$

where: VDV = vibration dose value $(m/s^{-1.75})$

$a(t)$ = frequency-weighted rms acceleration (m/s^2) over the frequency range 1 to 80Hz

T = total period of the day or night (in s) during which vibration can occur

The use of the fourth power method makes VDV more sensitive to peaks in the acceleration waveform. VDV accumulates the vibration energy received over the day-time and night-time periods. Acceptable values of vibration dose for intermittent vibrations are presented in table 3.2.

Vibration velocity can be used to estimate VDV. However, where possible, acceleration should be used when determining VDV.

Table 3.2 Vibration dose value ranges which might result in various probabilities of adverse comment within residential buildings (reproduced from BS 6472-1:2008)

Place and time	Low probability of adverse comment $(m/s^{-1.75})$	Adverse comment possible $(m/s^{-1.75})$	Adverse comment probable $(m/s^{-1.75})$
Residential buildings 16-hour day	0.2 to 0.4	0.4 to 0.8	0.8 to 1.6
Residential buildings 8-hour night	0.1 to 0.2	0.2 to 0.4	0.4 to 0.8

Note: For offices and workshops, multiplying factors of 2 and 4 respectively should be applied to the above VDV ranges for a 16-hour day.

BS 6472-1:2008 states that the VDV values in table 3.2 represent the best judgement currently available and may be used for both vertical and horizontal vibration, provided they are correctly weighted.

It will be noted that the VDV criteria have been presented as ranges rather than discrete values. This stems largely from the widely differing susceptibility to vibration evident among members of the population, and also from their differing expectation of the vibration. Some judgement will therefore have to be exercised when applying the VDV criteria.

With reference to table 3.2, adverse comment is not expected for VDV values less than 0.2 $(m/s^{-1.75})$ during the day-time and 0.1 $(m/s^{-1.75})$ during the night-time. Conversely, adverse comment is extremely likely for VDV values above 1.6 $(m/s^{-1.75})$ during the day-time and 0.8 $(m/s^{-1.75})$ during the night-time.

3.3 Effect of vibration on structures

Building components usually have residual strains as a result of uneven soil movement, moisture and temperature cycles, poor maintenance or past renovations and repairs. Therefore, even small vibration levels induced by nearby construction activity could trigger damage by 'topping up' residual strains. Consequently, it is difficult to establish a vibration level that may cause building damage and so controversy continues to surround the issue (Hunaidi 2000).

In addition to damage caused directly by vibration, indirect damage may result from differential movements caused by soil settlement due to densification. Loose sandy soils are particularly susceptible to densification when subjected to vibration.

Several countries have adopted standards for evaluating the effect of vibration on buildings. Again, Standards New Zealand has no current standard for providing guidance for evaluating potential building damage from ground-borne vibrations.

A review of international standards identified the following as being the most suitable for providing guidance as to possible building damage from mechanised construction activity:

- German Standard DIN 4150-3:1999 *Structural vibration – part 3: Effects of vibration on structures.*
- British Standard BS 7385-2:1993 *Evaluation and measurement for vibration in buildings, part 2. Guide to damage levels from ground-borne vibration.*
- Swiss Standard VSS-SN640-312a:1992 *Effects of vibration on construction.*

These three standards are expanded on below.

3.3.1 German Standard DIN 4150-3:1999

Whitlock (2010) found 'the use of DIN 4150-3:1999 is widespread in New Zealand and it has a history of successful implementation in projects that involve construction activities and/or blasting'.

The standard is written around the PPV metric and provides guideline values which, 'when complied with, will not result in damage that will have an adverse effect on the structure's serviceability'. For residential buildings, the standard considers serviceability to have been reduced if:

- cracks form in plastered surfaces of walls
- existing cracks in the building are enlarged
- partitions become detached from load bearing walls or floors.

These effects are deemed to be minor or superficial damage.

The guideline values are different depending on the vibration source and are separated on the basis of short-term and long-term vibration. The standard defines short-term vibration as ‘vibration which does not occur often enough to cause structural fatigue and which does not produce resonance in the structure being evaluated’. Long-term vibration is defined as all other types of vibration not covered by the definition of short-term vibration. In the context of road construction projects, Whitlock (2010) illustrates the difference between short- and long-term vibrations as follows:

> *‘… the short-term vibration definition would be applied to activities which follow the form of a single shock followed by a period of silence such as blasting, drop hammer pile-driving (ie non-vibratory), dynamic consolidation etc. All other construction activities can be considered as long term’.*

Guideline values for evaluating short-term and long-term vibration on structures from tables 1 and 3 of DIN 4150-3: 1999 have been combined in table 3.3, as in Whitlock (2010). The standard recognises commercial buildings can withstand higher vibration levels than residential and historic buildings. Also, the guideline values increase as the vibration frequency increases.

Table 3.3 Vibration guidelines from DIN 4150-3:1999 for assessing effects of vibrations on buildings

<table>
<tr><th rowspan="4">Type of structure</th><th colspan="5">Vibration thresholds for structural damage, PPV (mm/s)</th></tr>
<tr><th colspan="4">Short term</th><th>Long term</th></tr>
<tr><th colspan="3">At foundation</th><th>Uppermost floor</th><th>Uppermost floor</th></tr>
<tr><th>0 to 10 Hz</th><th>10 to 50 Hz</th><th>50 to 100 Hz</th><th>All frequencies</th><th>All frequencies</th></tr>
<tr><td>Commercial/industrial</td><td>20</td><td>20 to 40</td><td>40 to 50</td><td>40</td><td>10</td></tr>
<tr><td>Residential</td><td>5</td><td>5 to 15</td><td>15 to 20</td><td>15</td><td>5</td></tr>
<tr><td>Sensitive/historic</td><td>3</td><td>3 to 8</td><td>8 to 10</td><td>8</td><td>2.5</td></tr>
</table>

Note: When a range of velocities is given, the limit increases linearly over the frequency range.

The standard also contains criteria for buried pipework as a function of pipe material and the effects of vibration on floor serviceability. Regarding vibration-induced foundation settlement, the standard warns this can occur at vibration levels that are normally not expected to cause structural damage. For this to occur, the soil has to be very sensitive to vibration (as in non-cohesive, uniformly graded sand or silt for instance) and the vibration has to be continuous or frequent. Since few investigations have been made regarding dynamically induced settlement, the standard recommends that expert advice be sought whenever this is considered to be a possible issue.

3.3.2 British Standard BS 7385-2:1993

‘BS 7385-2:1993 sets vibration limits based on an extensive review of international case histories. The introduction states that despite the large number of UK case studies involved in the review, “very few cases of vibration-induced damage were found”’ (Whitlock 2010).

The guideline values in BS 7385-2:1993 are also in terms of PPV and have been summarised in table 3.4.

Table 3.4 Transient vibration guide values for cosmetic damage in BS 7385-2:1993

Building category	Type of building	Peak component particle velocity in the frequency range of predominant pulse	
		4Hz to 15Hz	15Hz and above
1	Reinforced or framed structures. Industrial and heavy commercial buildings.	50mm/s at 4Hz and above	
2	Unreinforced or light framed structures. Residential or light commercial type buildings.	15mm/s at 4Hz increasing to 20mm/s at 15Hz	20mm/s at 15Hz increasing to 50mm/s at 40Hz and above

Note 1. Values referred to are at the base of the building

Note 2. For building category 2, at frequencies below 4Hz, a maximum displacement of 0.6mm (zero to peak) should not be exceeded.

Comparing the guideline values in table 3.4 with those in table 3.3, it is apparent that the British standard is significantly less stringent than the German standard. Also there is no specific provision for historic or sensitive structures. These types of structures are addressed by the following clause in the British standard:

> *Important buildings which are difficult to repair may require special consideration on a case-by-case basis. A building of historical value should not (unless it is structurally unsound) be assumed to be more sensitive.*

'This approach to historic structures is quite different from that of DIN 4150-3:1999, which is less definitive with its definition of such buildings and more stringent in its criteria' (Whitlock 2010).

The standard also provides guidance regarding building damage due to soil compaction. Annex C of the standard states 'soils which have S-wave propagation velocities at around 100m/s start to become vulnerable at PPV values of about 10m/s'. Attention is drawn to loose and especially water-saturated cohesionless soils as these are particularly vulnerable to vibration, which may result in liquefaction.

3.3.3 Swiss Standard VSS-SN640-312a:1992

The Swiss Consultants for Road Construction Association (SCRCA) have developed a standard for assessing both transient (blasting) and continuous (other construction equipment) vibrations as shown in table 3.5 (Jackson et al 2007). The standard takes into account the type of construction and frequency of the vibration source. A category for historic structures is also included.

The vibration level guidelines given in the Swiss standard are regarded as being very conservative (Jackson et al 2007). Despite this, the Federal Transit Administration (FTA) has promoted their use during the environmental impact phase to identify potentially problematic locations (Hanson et al 2006).

Table 3.5 Swiss Standard VSS-SN640-312a construction vibration damage criteria (adapted from Jackson et al 2007)

Building class	Vibration source	Frequency range (Hz)	PPV (mm/s)
I	Machines, traffic	10 – 30	12.7
		30 – 60	–12.7 – 17.8
	Blasting	10 – 60	30.5
		60 – 90	–30.5 – 40.6
II	Machines, traffic	10 – 30	7.6
		30 – 60	–7.6 – 12.7
	Blasting	10 – 60	17.8
		60 – 90	–2.5 – 25.4
III	Machines, traffic	10 – 30	5.1
		30 – 60	–5.1 – 7.6
	Blasting	10 – 60	13.7
		60 – 90	–12.7 – 17.8
IV	Machines, traffic	10 – 30	3.0
		30 – 60	–3.0 – 5.1
	Blasting	10 – 60	7.6
		60 - 90	–7.6 – 12.7

Key for building class:

I Buildings of steel or reinforced concrete, such as factories, retaining walls, bridges, steel towers, open channels; underground chambers and tunnels with and without concrete lining

II Foundation walls and floors in concrete, walls in concrete or masonry; stone masonry retaining walls; underground chambers and tunnels with masonry linings; conduits in loose material

III Buildings as previously mentioned but with wooden ceilings and walls in masonry

IV Construction very sensitive to vibration; objects of historical interest

3.4 Suggested screening criteria for New Zealand conditions

The various standards discussed above are considered to be the most appropriate for assessing vibrations generated by mechanical equipment during road construction activities. For a more comprehensive review of vibration standards used in New Zealand for dealing with human comfort and cosmetic and structural damage to buildings, refer Whitlock (2010).

To identify projects that have the possibility of creating significant adverse impact, generalised data or predictive models are often used to provide estimates of ground-borne vibration levels. Such general assessments deal with the overall vibration velocity and, unlike physical measurements, do not consider the frequency spectra of the vibration.

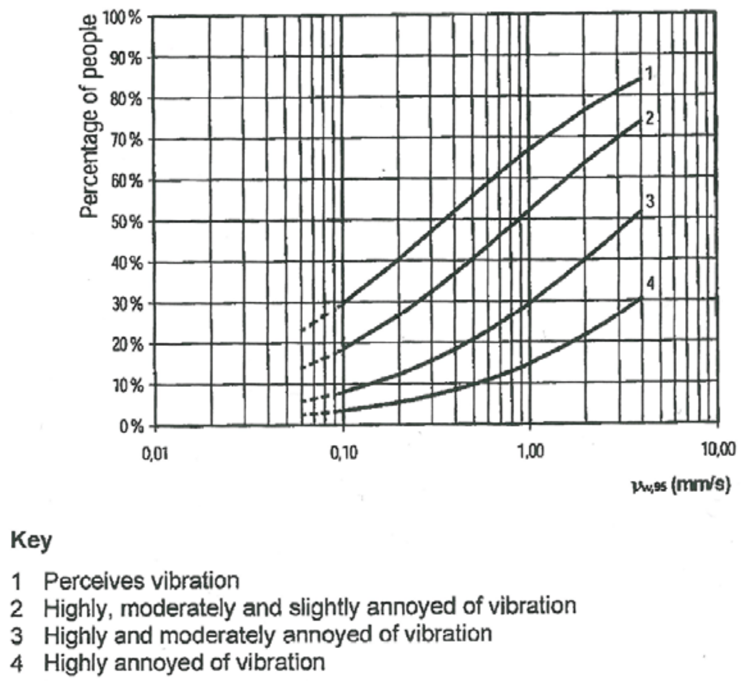
The criteria recommended for application to output from vibration models are:

- 0.5mm/s PPV for disturbance of building occupants
- 5mm/s PPV for building damage.

These values have been derived through consideration of the various standards discussed above. They should be regarded as interim only as the NZTA are in the process of developing state highway vibration assessment criteria, which the findings of this report will provide input to. Furthermore, these two criteria do not apply to soils susceptible to vibratory densification/liquefaction. In general, these soils comprise uniform grain size (ie low cohesion) silts below the water table and have a very low scala count, ie about two blows per 50mm. For such cases, specialist geotechnical advice should be sought as to what vibration levels could cause consolidation or densification of the soil, which is known to result in differential settlement and consequent building damage.

The Norwegian Standard NS 8176.E:2005 *Vibration and shock. Measurement of vibration in buildings from land based transport and guidance to evaluation of its effects on human beings* recognises there is a clear relationship between the magnitude of the vibrations and the number of people who respond to them. This relationship can be calculated and plotted in the form of exposure-effect curves as shown in figure 3.1.

Figure 3.1 The percentage of people with four levels of annoyance due to vibrations in dwellings, plotted against calculated statistical maximum values for weighted velocity, $v_{w,95}$ (NS 8176.E:2005)



Statistical analysis shows that the vibration effects from various sources are the same at a given vibration exposure. Therefore, the exposure-effect curves given in NS 8176.E:2005 do not distinguish among different sources of vibration. Furthermore, the curves show the average effect in the population.

Figure 3.1 can be used to quantify the effect of a vibration level of 0.5mm/s PPV on the occupants of a building in terms of perception and annoyance. For ground vibrations induced by construction equipment that has a dominant frequency ranging between 10–70Hz, NS 8176.E:2005 indicates that 10% of building occupants will be highly annoyed if the PPV measured is 0.5 mm/s. The standard also indicates that 56% of the building occupants will perceive a vibration at this level.

If the vibration level is increased by 100% from 0.5mm/s to 1mm/s PPV, the percentage of building occupants highly annoyed increases from 10% to 16%. The percentage of building occupants perceiving a vibration increases from 56% to 67%. This insensitivity to vibration level helps explain the large difference in human comfort criteria between the various standards and emphasises the highly subjective nature of the criteria recommended.

4 Measured vibrations

In making a quantitative assessment of vibrations from mechanised construction activities, the following three- step procedure is suggested (Hanson et al 2006):

1 Select equipment and from previously measured data determine the associated source levels for a known reference distance.

2 Make a propagation adjustment according to equation 2.12.

3 Apply the vibration annoyance and damage criteria from section 3.4.

To enable step 1 to be carried out, it was necessary to undertake experimental work as follows. Field data from construction sites throughout New Zealand was acquired for representative mechanised construction equipment operating on a range of soil types. The specific equipment monitored comprised three dozers, 12 rollers, one grader and one stabiliser. Photographs of these are provided in appendix A.

The equipment spanned weights from 4 tonnes to 33 tonnes and dominant operating frequencies from 12Hz to 64Hz.

4.1 Data acquisition

Ground-borne vibrations induced by the mechanised construction equipment were measured as accelerations in three orthogonal directions using two Syflex SF 3000L MEM triaxial accelerometers.

The distance between the accelerometers and the particular equipment generating the vibrations was continuously measured by having one global positioning system (GPS) unit attached to the closest triaxial accelerometer while the other GPS unit was attached to the particular equipment being monitored.

The GPS units were logged at one second intervals allowing the separation distance between the equipment and the measurement location to be determined as the difference in GPS readings made at the same instant.

The output from both triaxial accelerometers and the GPS unit attached to the triaxial accelerometer in closest proximity to the equipment was interfaced to a Measurement Computing Ltd USB – 608FS data acquisition card. The output from the trixial accelerometers was sampled at 0.001 second intervals (ie a sampling frequency of 1000Hz). The acquired acceleration and separation distance time histories were saved directly to the hard drive of a Toshiba Tecra laptop computer for later processing.

The time histories from each of the triaxial accelerometers were processed to give component peaks for acceleration and velocity as well as dominant frequency for each one second interval to enable matching with the GPS records.

Because the one second PPV for each of the three components were very similar in magnitude and because no one component was consistently dominant, the one second peak component velocities were additionally combined to provide a maximum vector sum for each triaxial accelerometer. This value is commonly known as the square root of the sum of squares (SRSS). The SRSS peak particle velocity is frequently reported in vibration studies as it provides a conservative estimate of the vibration level as the maximum of each component is used in the calculation regardless of the time when it occurs.

All measured ground vibrations presented in this report are therefore the maximum of the individual peak velocity components (ie PPV) or the square root of the squares of the peak velocities in the three orthogonal directions (SRSS values), both having units of mm/s.

4.2 Site descriptions

The following are brief descriptions of the sites including the soil type and construction happening at the time of the measurements and if dynamic cone (scala) penetrometer data was available for the site:

Napier – silty sand

This site was for a new interchange, the topsoil had been cleared and embankments were being built. The measurements were made with both the triaxial accelerometers and machinery on in situ soil. Scala penetrometer data was available.

MacKay's Crossing, Kapiti – peat

This site was part of a new road alignment. Measurements were made with the triaxial accelerometers on in situ peat and the machinery on a shallow layer of fill being placed over the peat. Scala penetrometer data was available.

MacKay's Crossing, Kapiti – greywacke fill

This site was part of a road realignment and consisted of approximately 2m of well compacted greywacke fill over weathered greywacke. There was also a thin layer of basecourse (approximately 200mm) over the fill which was being compacted. The measurements were made with the triaxial accelerometers on the fill and the roller on the basecourse over the fill. Scala penetrometer data was available.

Rotorua – ash

This site was a road realignment and all readings were away from the existing road. Both the accelerometers and the machinery were on in situ brown ash. An adjacent road cutting showed the ash layer to be at least 4m thick. Scala penetrometer data was available.

Rotorua – pumice fill

This was also part of the same road realignment; however, in this area a layer of pumice sand was being compacted over the ash. The pumice layer was approximately 3m thick and for the measurements both the accelerometers and the machinery were on the pumice fill. Scala penetrometer data was available.

Mt Roskill – sandstone

This site was for the construction of a new road. The measurement area consisted of a layer of weathered sandstone being compacted over in situ sandstone. The triaxial accelerometers were positioned where the fill was approximately 1m thick, whereas the machinery was operating on up to 4m of fill. No scala penetrometer data was available.

Greenhithe – sandstone

This site was for the construction of a new road. The triaxial accelerometers were on in situ weathered sandstone that had been cut, whereas the machinery was operating on a layer of basecourse over the weathered sandstone. No scala penetrometer data was available.

Foxton – dune sand

This site was for the reconstruction of an existing road. The triaxial accelerometers were positioned on in situ dune sand, while the machinery was operating on a layer of basecourse over dune sand. Scala penetrometer data was available.

Alpurt – sandy clay

The site was for the construction of a new road. Both the triaxial accelerometers and the machinery were on approximately 6m of recompacted sandy clay. No scala penetrometer data was available.

Alpurt – sandstone

The site was for the construction of a new road. Both the triaxial accelerometers and the machinery were on approximately 1m of hard sandstone fill over in situ hard sandstone. No scala penetrometer data was available.

Taupo – pumice sand

This site was a reconstruction of an existing road. For the measurements the triaxial accelerometers were on in situ pumice sand and the machinery was on a layer of basecoarse being compacted over the pumice sand. Limited scala penetrometer data was available.

Christchurch – silt

This site was for a new subdivision. The triaxial accelerometers were on in situ silt, while the machinery was on a layer of basecourse over silt. No scala penetrometer data was available.

Rotorua city – volcanic/sandy loam

This site was from a previous project and was included because of the availability of scala penetrometer data. The soil is described as sandy loam of volcanic origin. The triaxial accelerometers were on in situ ground and the truck was on a sealed road. The attenuation was calculated from the difference between the two triaxial accelerometers and so relates only to the in situ soil.

4.3 Vibration measurements

SSRS peak particle velocity measurements of ground vibrations were made for a variety of mechanised road construction equipment at each of the sites visited. Photographs of each piece of equipment monitored are provided in appendix A.

The SRSS values have been normalised to a separation distance of 10m using equation 2.12 so that direct comparisons can be made. These normalised SRSS values and the associated dominant frequency of the induced ground vibrations are presented in table 4.1.

With reference to table 4.1, ground vibrations induced by passing vehicles are a function of vehicle mass, vehicle speed and roughness of the road. The SRSS value of 0.95mm/s PPV tabulated for the truck and trailer unit is therefore for the following specific conditions:

- mass of vehicle = 44 tonnes
- speed of travel = 50km/h
- average lane roughness of road in vicinity of vibration measurements = 96 NAASRA counts/km or 3.67m/km IRI.

4.3.1 Findings for mechanised road construction equipment

With reference to the 10m normalised SRSS values in table 4.1, it can be seen that the measured vibration levels span a wide range from 0.4mm/s PPV to a maximum of 11.0mm/s PPV, which is more than twice the screening criterion of 5mm/s PPV, suggested in section 3.4, for damage to residential buildings. It will also be noted that, even for quite similar types of machinery, the range in vibration levels can be wide. For example, the 10m normalised SRSS values for rollers ranged from 0.4mm/s PPV to 6.95mm/s PPV.

There was an expectation that the softer soils would produce higher amplitude vibrations but this was not always the case. Also contrary to expectation, there was no significant correlation between weight of the equipment and the resulting ground vibration level, with the heaviest piece of machinery, a 33 tonne static roller, inducing one of the lowest ground vibration levels (0.7mm/s PPV).

Table 4.1 Measured mechanised construction equipment induced ground vibrations

Site	Soil type	Type of equipment	Equipment Item	Equipment weight, (kg)	Measured attenuation coefficient α	Frequency (Hz)	Frequency independent soil property, ρ (s/m)	SRSS @ 10m (mm/s, PPV)	Scala Ave/max	Nominal Wavelength (m)
Napier	Silty sand	Dozer	Komatsu D65E	20280	0.009	14	2.04×10^{-4}	3.75	5.6 / 10	21
Napier	Silty sand	Roller	Hamm 3410	11015	0.029	29	3.18×10^{-4}	6.95	5.6 / 10	10
McKays	Peat	Dozer	Caterpillar D4H	7855	0.073	12	1.94×10^{-3}	11.9	1.3 / 2	17
McKays	Peat	Roller	Bomag BW177D3	7230	0.073	28	8.30×10^{-4}	0.8	1.3 / 2	7
McKays	Greywacke fill	Roller	Dynapac CC422HF	10400	0.088	64	4.38×10^{-4}	1.13	5.5 / 7	5
Rotorua	Ash	Excavator	Komatsu PC60	6000	-	21	-	1.8	6.9 / 14	14
Rotorua	Pumice fill	Roller	Sakai SW500	4000	0.193	50	1.23×10^{-3}	2.9	2 / 4	4
Rotorua	Pumice fill	Excavator	Sumitomo SH120	12600	0.061	20	9.71×10^{-4}	5.4	2 / 4	10
Rotorua	Pumice fill	Dozer	Komatsu DP31P	7300	0.031	12	8.22×10^{-4}	3.8	2 / 4	17
Mt Roskill	Sandstone	Roller	Caterpillar 815B	20755	0.025	13	6.12×10^{-4}	1.9	-	15
Mt Roskill	Sandstone	Roller	Caterpillar CS663E	17100	0.012	26	1.47×10^{-4}	1.8	-	19
Mt Roskill	Sandstone	Roller	Caterpillar 825C	32734	0.009	13	2.20×10^{-4}	0.7	-	23
Greenhithe	Sandstone	Roller	Caterpillar CB544	10700	0.070	40	5.57×10^{-4}	1.1	-	8
Foxton	Dune sand	Roller	Sakai SW70C	7050	0.016	53	9.61×10^{-5}	0.4	7.1 / 11	9
Alpurt	Sandy clay	Roller	Caterpillar CP663E	16800	0.158	28	1.80×10^{-3}	3.55	-	7
Alpurt	Sandstone	Roller	Dynapac CA602	18600	0.046	28	5.23×10^{-4}	12.4	-	7
Taupo	Pumice sand	Roller	Hamm 3414	14340	0.012	37	1.03×10^{-4}	9.0	6.6 / 14	5
Taupo	Pumice sand	Grader	Volvo G726B	16000	-	43	-	0.9	-	5
Taupo	Pumice sand	Stabiliser	Wirtgen WR2000	22000	-	36	-	0.9	-	6
Christchurch	Silt	Roller	Sakai SW70C	7050	0.037	53	2.22×10^{-4}	0.4	-	6
Rotorua city	Volcanic/ sandy loam	Truck & trailer unit	Hino	44000	0.018	23	2.49×10^{-4}	0.95 (50km/h, 3.7 m/km IRI)	5.6 / 8	13

5 Pile driving

Piling operations have been considered separately from other mechanised construction activities because of their intrusive nature and potential to cause damage to adjacent structures. Furthermore, piling operations differ from many other vibration sources in that the position of the source that transfers energy into the ground continually changes as piling progresses. This is because the tip of the pile encounters different soil and the length of the pile shaft in contact with the ground increases as the pile is driven progressively deeper.

Two methods are commonly used in New Zealand for driving piles into soil to provide foundation support for buildings or other structures, these being the impact or recursive hammer and the vibratory hammer.

Impact hammers work by dropping a large weight of several tonnes onto the end of the pile in order to drive the pile into the ground. Hydraulics, or a combustion process which essentially replicates a very large two-stroke diesel engine, are employed in raising the weight. Hydraulic piling hammers are more environmentally acceptable than the older, less efficient diesel and air piling hammers as they generate less noise and pollutants. Impact hammers produce intermittent vibrations.

Vibratory hammers work by imparting a vertical vibration onto the pile through a system of counter-rotating eccentric weights, powered by hydraulic motors, and are designed in such a way that horizontal vibrations cancel out while vertical vibrations are transmitted into the pile. Vibratory piling hammers can either drive in or extract a pile. They are often chosen to mitigate noise, for example when the construction is very close to occupied buildings, or when there is not sufficient vertical clearance above the foundation to permit use of a conventional impact hammer. Vibratory hammers are available with several different vibration rates, ranging from about 1200 vibrations per minute (VPM) to about 2400 VPM. Vibratory hammers are particularly effective at driving piles in soils that are easily mobilised, such as well sorted sand. Vibratory hammers generate continuous vibrations.

The schematics in figure 5.1 illustrate the differences between the two main types of hammers, vibratory (labelled a) and impact (labelled band c).

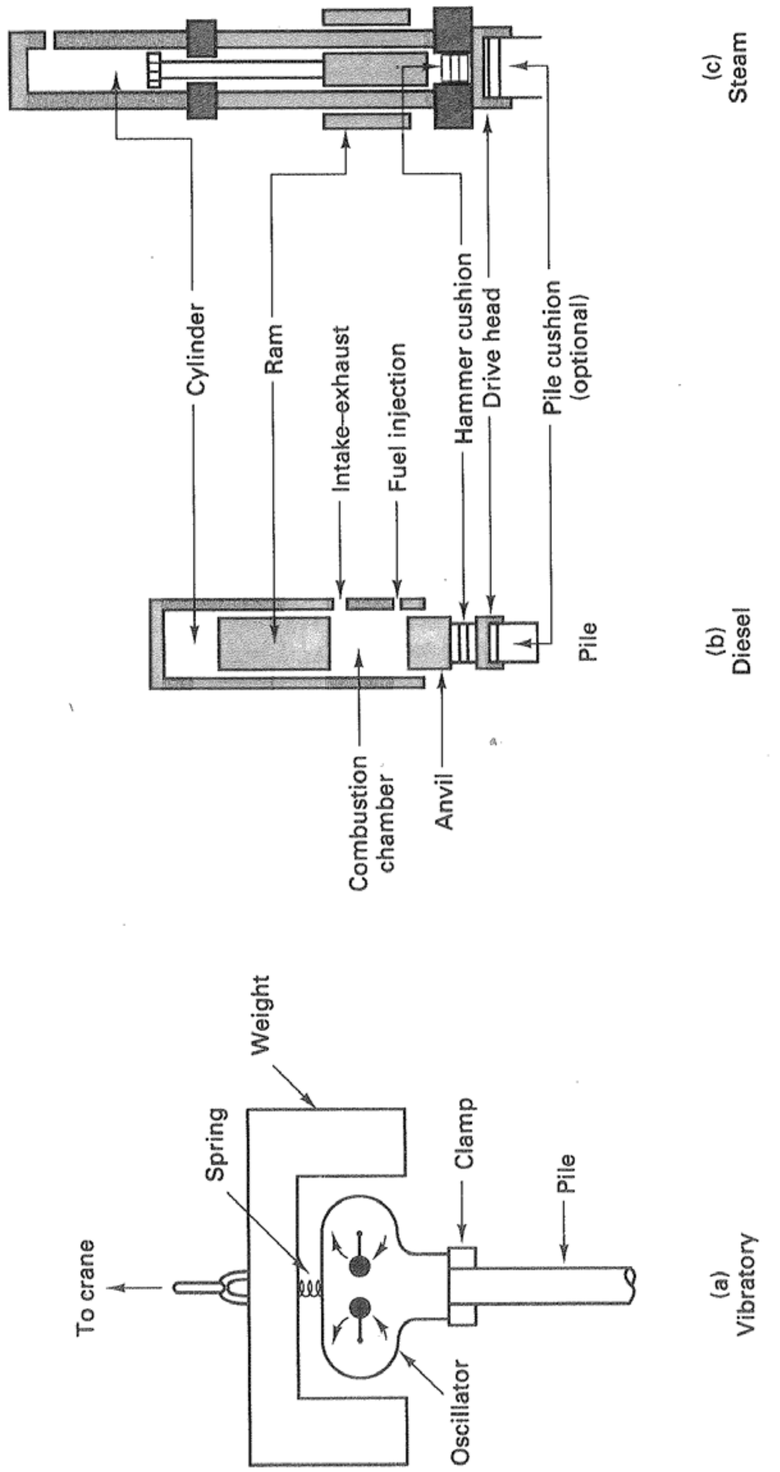
In the sections below, empirical prediction methods are presented and discussed in the context of data acquired from four New Zealand case studies involving two separate impact hammer and vibratory hammer piling operations. The reader is referred to Hiller and Crabb (2000) for a comprehensive review of ground-borne vibrations arising from piling works.

5.1 Impact hammers

The mechanism by which an impact hammer induces ground-borne vibrations is the same as drop-weight dynamic soil compaction. Dowding (2000) presents a summary of vibration measurements for a series of dynamic compaction related case studies. The maximum vibration levels from these case studies are graphed in figure 5.2 below for both loose and stiff soils. These vibration levels can be regarded as covering the maximum and minimum expected from impact hammer piling.

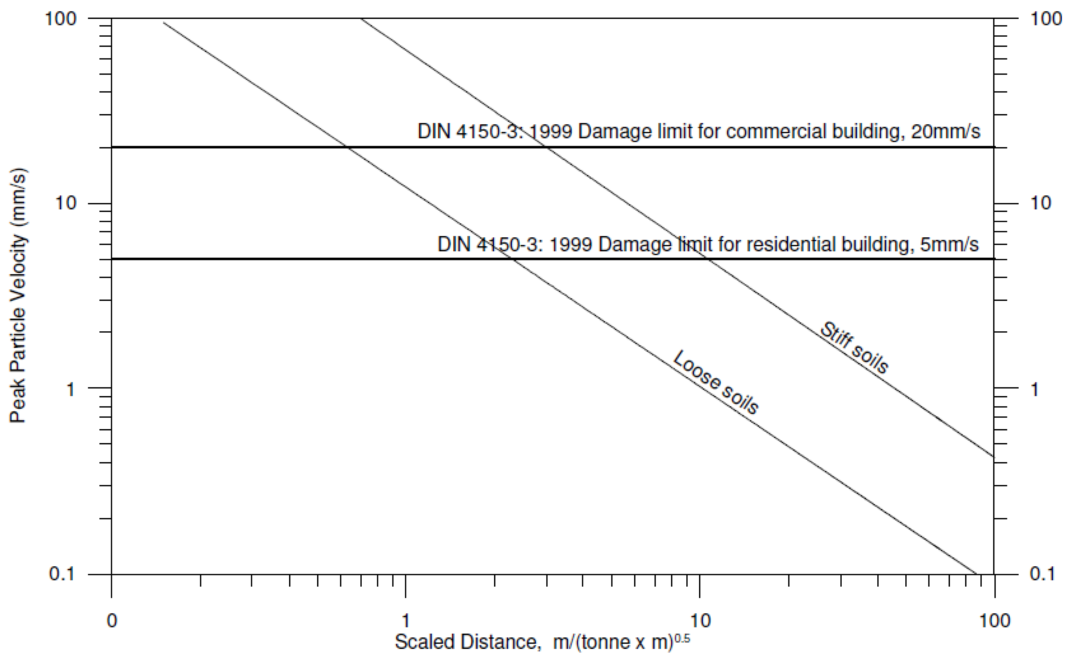
During the dynamic compaction process, the soil will progress from loose to stiff, but at any position the maximum expected vibrations will relate to the stiff soil line shown in figure 5.2. Similarly with piling, the pile is likely to be founded on a stiff layer. However, as this stiff layer may be deep, the vibration at the surface may be less than indicated in figure 5.2.

Figure 5.1 Components of three principal types of hammers (Dowding 2000)



With reference to the horizontal ('x') axis of figure 5.2, the scaled distance is the distance from the source divided by the square root of the drop height multiplied by the drop weight. Scaled distance is a convenient method for comparing energies from different energy source configurations.

Figure 5.2 Vibration envelopes for dynamic compaction of loose and stiff soils (after Dowding 2000)



An expected value of ground vibration level in terms of PPV can be calculated for a particular drop-weight hammer or dynamic compaction setup using equation 5.1, which has been derived for stiff soils (Dowding 2000).

$$PPV = 67\left(\frac{r}{\sqrt{mh}}\right)^{-1.1} \qquad \text{(Equation 5.1)}$$

where: *PPV* = peak particle velocity (mm/s)

r = the radial distance from the source (m)

m = the drop mass (tonnes)

h = the drop height (m)

An advantage of equation 5.1 is that its application requires no knowledge of surrounding soil type and associated attenuation factor.

The predictive equation given in BS 5228-2:2009 for estimating ground-borne vibrations from impact hammer piling is reproduced below as equation 5.2.

$$PPV = k_p\left(\frac{\sqrt{W}}{{r_s}^{1.3}}\right) \qquad \text{(Equation 5.2)}$$

where: *PPV* = peak particle velocity (mm/s)

k_p = scaling factor from table 5.1

W = nominal hammer energy, in joules (J)

r_s = the slope distance from the pile toe (m) which is the vector sum of pile toe depth and distance measured along the ground surface

Unlike equation 5.1, equation 5.2 takes into account the length of pile below the ground surface, the soil type the pile toe is being driven through, and whether or not the pile is at refusal. Refusal is the condition when the effective energy of the hammer is no longer sufficient to cause penetration (the hammer is too light or velocity at impact too little).

Table 5.1 Values of k_p scaling factor use with equation 5.2 (from BS 5228-2:2009)

Ground conditions	Value of 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 fills	1.5
Pile toe being driven through: • soft cohesive soils • loose granular soils • loose fill • organic soils	1

5.1.1 Case studies

Two case studies were considered and figure 5.3 shows the measured vibrations. Case study 1 involved pile driving operations that took place as part of the construction of the Meeanee Interchange, Napier, in which an impact hammer with a drop mass of 14 tonnes and a drop height of 0.45m was employed. Case study 2 involved the driving of steel casings at Paremata for ground improvement. In this case, an impact hammer with a drop mass of 8 tonnes and a drop height of 0.8m was employed.

Superimposed on figure 5.3 are the estimated vibrations for both piling operations using equation 5.1and BS 5228-2:2009 (equation 5.2) for the following three soil conditions:

- soft cohesive soils (k_p = 1)
- very stiff cohesive soils (k_p = 3)
- pile driven to refusal (k_p = 5).

Also superimposed on figure 5.3 are the suggested screening criteria from section 3.4 of 0.5mm/s PPV for annoyance and 5mm/s PPV for damage.

With reference to figure 5.3, it appears that soil conditions were different for the two case studies with the piles being near to refusal in case study 1 and being driven through stiff to very stiff cohesive soil in case study 2. This may explain why vibration levels estimated using equation 5.1 were significantly less than observed for case study 1 but in very good agreement with the vibration levels observed for case study 2.

The majority of the measured ground vibrations fell within the envelope created by using equation 5.2 (ie BS 5228-2:2009) with k_p equal to 1 (lower bound) and 5 (upper bound). ***Therefore, for conservative estimates of ground vibrations induced by impact hammer piling, it is recommended that equation 5.2 be used with k_p set to 5 in preference to equation 5.1.***

Figure 5.3 Comparison of measured and predicted impact hammer-induced ground-borne vibrations

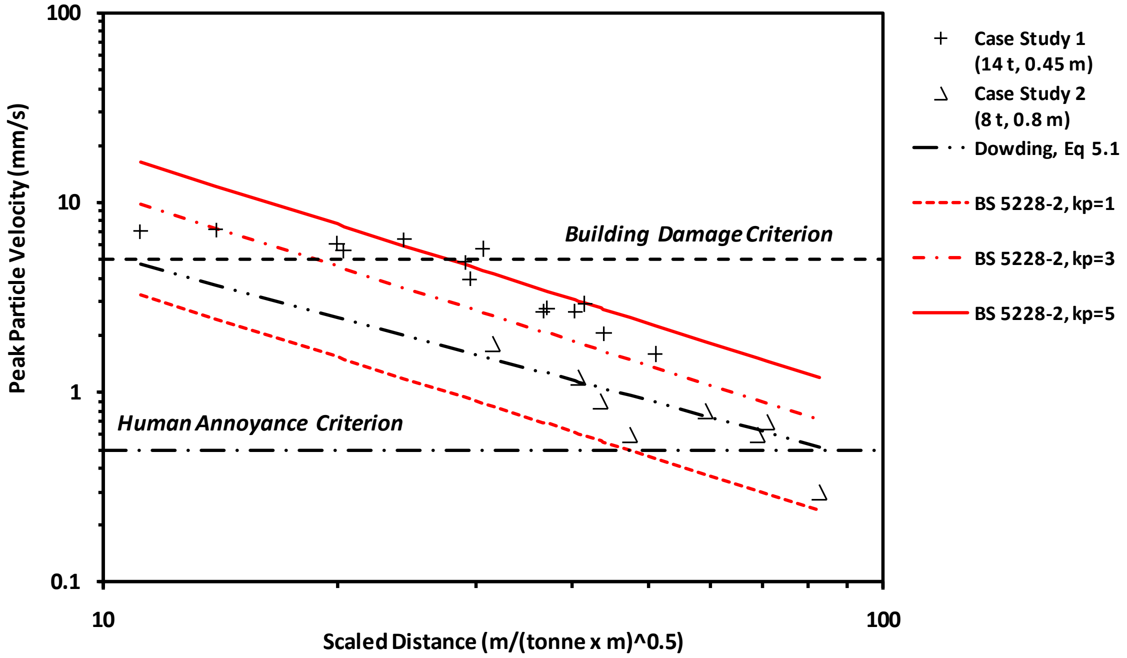


Figure 5.3 also indicates that the scaled distance must be greater than 28 $m/\sqrt{tonne.m}$ to minimise the likelihood of impact hammer piling operations causing cosmetic damage to nearby buildings.

5.2 Vibratory pile driving

Operating frequencies of vibratory pile drivers/extractors typically range from 20Hz to 40Hz. The vibration generated reduces the shear strength of the soil close to the pile being driven, thereby reducing the friction and the pile-soil interface. The combined weight of the pile and the driver cause the pile to be driven into the ground.

With reference to table 2.8, the magnitude of ground-borne vibrations induced by vibratory hammers are typically a quarter to half those induced by impact hammers.

BS 5228-2:2009 provides an empirical equation, reproduced as equation 5.3, for predicting the resultant PPV for vibratory piling operations that has a 5% probability of being exceeded.

$$v_{res} = \frac{266}{x^{\delta}} \qquad \text{(Equation 5.3)}$$

where: v_{res} = resultant PPV (mm/s)

x = distance measured along the ground surface (m)

δ = 1.2 (start up and run down)

= 1.3 (all operations)

= 1.4 (steady state operations)

5.2.1 Case studies

Two case studies were available for facilitating comparisons of measured ground-borne vibrations from vibratory pile operations with estimates derived from equation 5.3.

Case study 1 involved dynamic sheet piling operations along the Waiwhetu Stream, Gracefield. In this case, an international construction equipment (ICE™) 416L driver/extractor was used. Tri-axial accelerometer measurements of ground vibrations were made adjacent to a stream bank with some measurements made on the bank itself and others on the stream bed. The magnitude of the vibrations was found to be similar at both these locations.

Case study 2 involved sheet piles driven by an ICE™ vibro hammer (216) and pile casings driven by an ICE™ 416 driver/extractor in Matatua Road, Raumati Beach. Again, tri-axial accelerometers were used to measure the ground vibrations induced during the vibratory piling operations being undertaken with these two pieces of equipment.

Technical specifications of the ICE™ vibratory hammers used in both case studies are summarised in table 5.2.

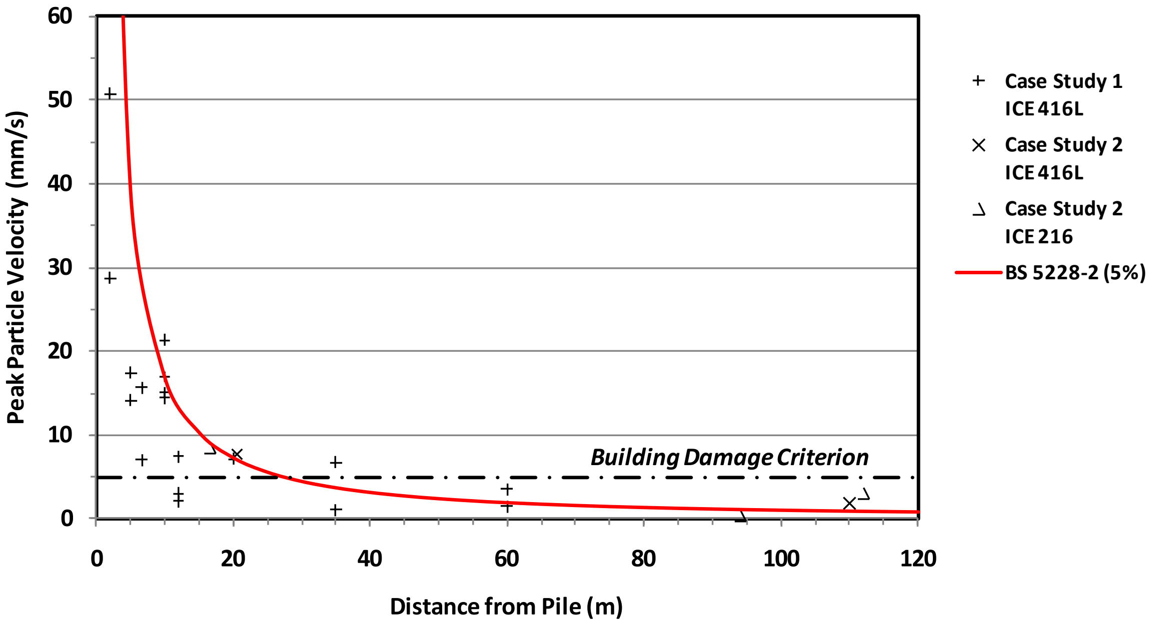
Table 5.2 Key specifications of case study vibratory hammers

Parameter	Model of ICE™ vibratory hammer	
	Vibro hammer (216)	416L
Centrifugal force (kN)	325	645
Vibrating weight (kg)	1.48	7
Engine power (kW)	116	260
Eccentric moment (kg-m)	11.5	23
Max. amplitude (mm)	16	14
Max. frequency (Hz)	26.7 (1600RPM)	26.7 (1600RPM)

Figure 5.4 shows the measured vibrations for the two vibratory pile driving case studies. For case study 1, the measured component vibrations were of similar magnitude and so were combined using the square root of the sum of squares (SRSS) method and this has been graphed in figure 5.4. The measured component vibrations were also of similar magnitude for case study 2. However, they have been graphed in figure 5.4 as the maximum component PPV for each measurement location.

Superimposed on figure 5.4 are the estimated maximum resultant vibrations calculated by equation 5.3 with δ set to 1.2 and the suggested screening criteria of 5mm/s PPV for damage. With reference to figure 5.4, the theoretically derived curve of maximum resultant vibration levels fits the measured data well up to a distance of 120m from the source. It is also apparent from figure 5.4 that a separation distance of 27m or greater between the vibratory piling operations and a building is required to ensure ground-borne vibrations are below the criterion value of 5mm/s PPV for the onset of cosmetic building damage.

Figure 5.4 Comparison of measured and predicted vibratory hammer-induced ground-borne vibrations



6 Site-specific attenuation and its prediction

With reference to section 2.1.4, the soil attenuation characteristics determine the rate at which ground borne vibrations decay with distance from source. Therefore, the opportunity was taken to examine the soil attenuation coefficient (α) and associated frequency independent material property (ρ) at the 10 sites where vibrations induced by the mechanised construction equipment were measured (refer chapter 4) to establish how these two parameters vary with common New Zealand soil types. Scala test results for the sites were also considered to determine if a correlation between the soil strength and attenuation coefficient exists.

6.1 Attenuation curves from measured values

Attenuation curves were generated by plotting the measured SRSS values against the time synchronised distance the triaxial accelerometers were from the mechanised construction equipment being monitored with GPS. Equation 2.9, with γ set to 0.5, was fitted to these attenuation curves to obtain estimates of soil attenuation coefficient, α, for all the construction sites where the ground vibration measurements were made.

This approach assumes that the source vibration is constant throughout monitoring period. This is a reasonable assumption for vibrating rollers. However, for excavators and dozers, the induced ground vibrations are much more varied and so in these cases the attenuation curve was usually fitted to the upper envelope of the data.

With reference to table 4.1, the derived α are tabulated along with the dominant frequency of the ground vibration. The frequency independent material property of the soil, ρ, was able to be derived by dividing α by the product, $\pi \times frequency(Hz)$, as per equation 2.11.

The derived values of ρ ranged from 9.61×10^{-5} (s/m) to 1.94×10^{-3} (s/m), which agree well with the values given in table 2.7 for class 1 and 2 soils covering weak or soft soils through to competent soils (ie silty clays, gravel, silts and sandy clays).

Also tabulated in table 4.1 is the nominal ground vibration wavelength for each construction site monitored. This was calculated by dividing the soil S-wave velocity, estimated from sources such as table 2.2, by the dominant vibration frequency. The calculated wavelengths ranged from 5m to 23m.

The derived values of ρ for a particular construction site were of similar magnitude irrespective of the source vibration, as indicated by the ranges tabulated table 6.1. This result supports Amick's (1999) premise that attenuation of ground-bourne vibrations induced by construction activity is frequency-dependent and so any vibration assessment must consider frequency as well as amplitude of vibration.

The observed variation in ρ at a construction site can, in part, be explained by the higher frequency vibrations having shorter wavelengths than the lower frequency vibrations. As a consequence, the higher frequency vibrations are more influenced by the shallow soil layers, which tend to have a higher intensity of jointing (ie fractures or cracks) than the deeper soil layers influencing lower frequency vibrations. This jointing contributes to the inherent variability of soil characteristics.

Table 6.1 Summary of measured ranges of the frequency independent soil property, ρ, for various New Zealand soil types

Description of soil	Frequency independent soil property, ρ (s/m)	Attenuation coefficient, α, at 5Hz (m^{-1})
Silty sand	2.04×10^{-4} – 3.18×10^{-4}	0.003 – 0.005
Sandstone	1.47×10^{-4} – 6.12×10^{-4}	0.002 – 0.01
Pumice fill	8.22×10^{-4} – 1.23×10^{-3}	0.013 – 0.019
Peat	8.30×10^{-4} – 1.94×10^{-3}	0.013 – 0.03

6.2 Comparison with scala penetrometer

A review of the literature failed to find any reported studies that investigated the degree of correlation between soil attenuation coefficients as given in tables 2.4 to table 2.6 and soil tests commonly used in geotechnical investigations. This is somewhat surprising as with reference to table 2.4, weaker or softer soils generally have larger magnitude soil attenuation coefficients than the harder/stronger soils.

As discussed earlier, a high percentage of the vibration energy from construction equipment is expected to propagate as R-waves. It has been also shown that most of the energy of the R-wave is confined to within one wavelength deep below the surface, with the maximum amplitude at one third of a wavelength deep. Therefore, it is logical that the attenuation measured at a particular site will be most influenced by the soil profile over two-thirds to one wavelength deep below the surface.

Most soil tests routinely performed at road construction sites are spot readings at the surface, such as density (via a nuclear densometer) or deflection (via the Loadman™, a portable FWD). However scala penetrometer tests used for determining the penetration resistance of a soil are also common and include a larger sample of the soil profile. Accordingly, it was decided to use scala penetrometer readings, where possible, to examine their relationship with measured soil attenuation coefficients.

Of the 10 sites where attenuation measurements were made, scala penetrometer tests were obtained for seven. Some of these scala penetrometer tests were carried out at the time of the attenuation measurements while others were carried out sometime prior.

Although wavelengths were not directly measured, equation 2.5 was used to infer wavelength values by using actual measurements of the dominant frequency of the ground vibrations and a R-wave velocity assumed appropriate for the soil type at the measurement site. The calculated wavelengths are in the order of 4m to 17m depending on the type of construction equipment being investigated. The maximum depth of the scala penetrometer testing ranged between 1.1m and 3m, corresponding approximately to between 0.1 and 0.8 of a wavelength. The resulting wavelength estimates have been tabulated in table 4.1 together with the average and maximum scala penetrometer readings in blows/50mm.

The measured soil attenuation coefficients have been plotted against both the average and maximum scala values in figures 6.1 and 6.2 respectively. The plotted values of soil attenuation coefficient have been normalised to a frequency of 5Hz to enable values obtained from the various construction equipment to be directly compared. Superimposed on figures 6.1 and 6.2 are the ranges of attenuation coefficient tabulated in table 2.4 for the main soil types with the band labelled '1' corresponding to weak or soft soils, band labelled '2' corresponding to competent soils and band labelled '3' corresponding to hard soils.

Figure 6.1 Relationship between attenuation coefficient (α) (normalised to 5Hz) and maximum scala penetrometer reading ($y = 0.0351e^{-.236x}$, $R^2 = 0.888$)

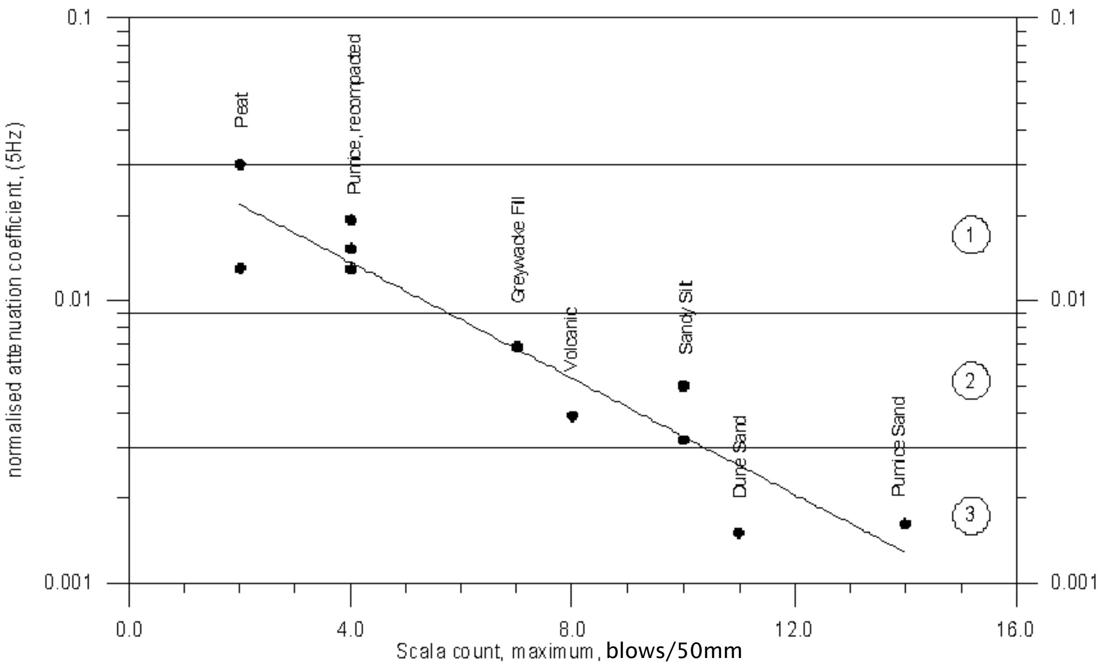
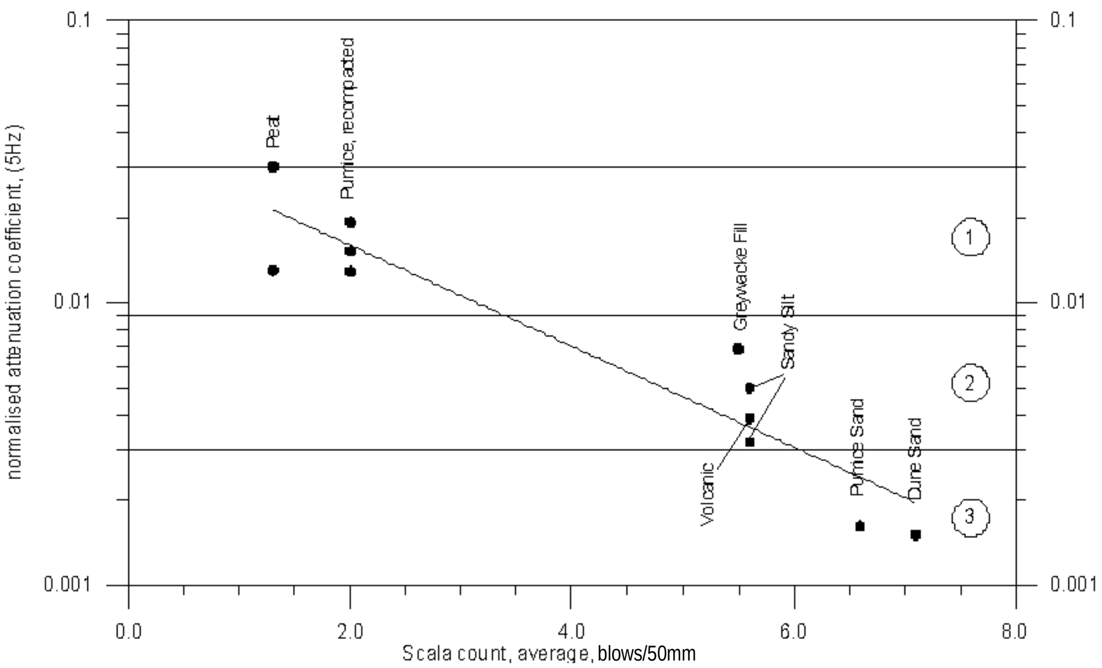


Figure 6.2 Relationship between attenuation coefficient (α) (normalised to 5Hz) and average scala penetrometer reading ($y = 0.0364e^{-.411x}$, $R^2 = 0.886$)



With reference to figures 6.1 and 6.2 the attenuation coefficients determined for the various New Zealand soil types fit well inside the ranges expected for the particular soil classification. Also of note is that the results for the pumice and dune sands followed the trend of decreasing attenuation with increasing scala count. This result suggests that at the time the measurements were made, the pumice and dune sands were behaving more like a hard soil.

Both average and maximum scala readings are shown to be very good predictors of soil attenuation, even for volcanic soils, with a coefficient of determination (r^2) of about 0.9 being achieved. Therefore, based on the above graphs, reasonable estimates of soil attenuation coefficient can be had by using either equation 6.1 or 6.2 below.

$$\alpha(5Hz) = 0.0351e^{-0.236SCALA_{max}} \quad \text{(Equation 6.1)}$$

$$\alpha(5Hz) = 0.0364e^{-0.411SCALA_{avg}} \quad \text{(Equation 6.2)}$$

where: $\alpha(5Hz)$ = soil attenuation coefficient (m^{-1}) for a frequency of 5Hz

$SCALA_{max}$ = maximum scala reading (blows/50mm)

$SCALA_{avg}$ = average scala reading (blows/50mm)

Cautionary note:

Scala penetrometer readings reflect seasonal conditions, ie more blows per 50mm can be expected in summer than winter. Therefore, when investigating the relationship between scala penetrometer readings and measured soil attenuation coefficients, it is important that the scala penetrometer readings are made as close to the time of the soil attenuation measurements as possible. However, when calculating soil attenuation coefficients from scala penetrometer readings, a seasonally averaged value should ideally be employed so that the result is more representative.

7 Site assessment using non-destructive testing

The use of the falling weight deflectometer (FWD) to identify vibration sensitive work zones has been reported by Jackson et al (2008). Despite being site specific, this method does not require detailed knowledge of the site geology and therefore soil attenuation characteristics as attenuation is automatically accounted for. The basis of the method is described below.

7.1 Predictor curve

The method described by Jackson et al (2008) utilises displacement time histories from FWD tests to generate site-specific ground vibration prediction curves of the form shown in figures 5.1 and 5.2. Because the R-wave does not fully form until some finite distance from the impact source, the displacement time history at the centre of the FWD load plate is not used in the analysis.

The analysis steps are as follows:

1. Particle velocity time histories at each FWD geophone sensor location are derived by differentiating the displacement time history:

$$V(t) = \left(\frac{dx(t)}{dt}\right) \qquad \text{(Equation 7.1)}$$

where: *x(t)* = the FWD displacement time history at a particular transducer

The peak particle velocity (PPV) at each FWD transducer is determined from:

$$PPV = max\ (V(t)) \qquad \text{(Equation 7.2)}$$

The scaled distance (SD) of the FWD transducer is given by:

$$SD = \frac{D}{\sqrt{F \times z}} \qquad \text{(Equation 7.3)}$$

where: *D* = the distance from the centre of the load plate to the FWD transducer of interest

F = the FWD force (mass × gravity) and z displacement (drop height, m)

The scaled distances of the FWD are subsequently plotted versus the PPV on a log-log plot. The data from all the FWD drops should be plotted excluding the transducer directly under the loading plate. Non-linear regression is used to fit a power curve to the resulting data of the following form:

$$PPV_{predicted} = b \times (SD)^m \qquad \text{(Equation 7.4)}$$

where: *b* and *m* are regression constants.

To ensure a conservative prediction of PPV, the 95% confidence interval is applied to the regression constant b:

$$PPV_{predicted} = (b + 1.96SE) \times (SD)^m \qquad \text{(Equation 7.5)}$$

where: SE is the standard error of estimation

7.1.1 Equipment energy

In order to apply the site characterisation method using FWD data, the amount of energy transferred to the ground must be able to be estimated. Therefore, the method is particularly suited to predicting vibration levels from impact-related construction activities, such as vibratory compaction, dynamic compaction and drop weight piling. The procedure for calculating the energy generated by these two activities is described below.

7.1.1.1 Vibratory compaction

Jackson et al (2008) present an expression for calculating the energy produced by a vibratory roller. To evaluate the expression some basic properties of the vibratory roller are required. These include the peak load, peak amplitude, operating frequency and drum diameter. The expression is shown below:

$$W_{roller} = k d_{drum} n P A \quad \text{(Equation 7.6)}$$

where: W_{roller} = the energy of the roller

k = calibration constant assumed to be 1

d_{drum} = drum diameter

n = number of blows per unit distance

P = peak load

A = peak amplitude

The scaled distance, SD (equation 7.3) can then be found by multiplying the scale factor by the required distance, D, from the vibratory roller:

$$SD = \frac{D}{\sqrt{W_{roller}}} \quad \text{(Equation 7.7)}$$

where: D = distance from roller to location of interest

For example, the distance, D, could be the distance between the roller and a building or structure.

The scaled distance from equation 7.7 is input into the site-specific predictor curve (equation 7.5), to calculate the expected vibrations at the building or structure to determine if they satisfy the suggested criteria given in section 3.4 and/or determine the critical separation distances to ensure threshold limits for human comfort and structural damage are not exceeded.

7.1.1.2 Drop-weight piling or dynamic soil compaction

The procedure outlined for determining site-specific vibration levels generated by a vibratory roller at a particular location can also be applied to impact hammer piling or drop-weight dynamic soil compaction. However, equation 7.6 is replaced by equation 7.8 below to calculate the energy produced:

$$W = mgh \quad \text{(Equation 7.8)}$$

where: W = the energy of the equipment (kJ)

m = the drop mass (tonnes)

g = gravity = 9.81 m/s^2

h = the drop height (m)

8 Vibration assessment guidelines

The operation of construction equipment causes ground vibrations that spread through the ground and diminish in strength with distance. Buildings and structures in the vicinity of the construction site respond to these vibrations with varying results ranging from no perceptible effects at the lowest levels, perceptible vibrations at moderate levels and slight damage at the highest levels (Hanson et al 2006). Some examples of construction equipment that generate few or no ground vibrations are air compressors, light trucks and hydraulic loaders, whereas construction activities that typically generate the most severe vibrations are pile-driving, vibratory compaction, and drilling or excavation in close proximity to vibration sensitive structures.

As construction activity can result in varying degrees of ground vibration, the first step in determining whether the resulting vibrations will be problematic from the perspective of occupant annoyance or building damage is to undertake a qualitative assessment. This qualitative assessment comprises a desk study, which considers the age of surrounding structures, their condition and historic importance, soil attenuation characteristics, proposed construction works and their duration and proximity to pertinent structures (both underground and above ground).

In cases where it has been assessed there is significant potential for prolonged annoyance or building damage from construction vibrations, a quantitative assessment should be carried out to better estimate the impact of the vibrations so that appropriate ameliorating measures can be taken. The most accurate way of determining a site's response to the planned construction activity is to make trial vibration measurements at the site. This involves operating the actual construction equipment or its equivalent at locations that cover all the soil conditions for the site but are well away from any buildings. Should this not be feasible, the following three methodologies can be employed to better quantify the vibration impacts:

1 Previously measured vibration source levels in combination with site scala penetrometer results

2 Empirical formulae given in BS 5228-2:2009

3 FWD derived predictor curves.

These three methodologies are described in greater detail in section 8.1.

Once the construction vibration has been assessed quantitatively using the above methodologies, a vibration management plan covering equipment location and processes needs to be prepared. The key elements of a vibration management plan are covered in section 8.2.

8.1 Vibration assessment methodologies

8.1.1 Assessment using previously measured source vibrations

This methodology is recommended for equipment associated with site preparation, ie excavation, levelling, trenching and compaction of fill. It involves selecting from table 4.1 the appropriate equipment and the associated vibration source levels at the reference distance of 10m. A propagation adjustment is then made as follows:

1 Use the maximum or average scala penetrometer readings for the construction site (equations 6.1 and 6.2) to calculate the soil attenuation coefficient for a frequency of 5Hz, ie $\alpha(5\text{Hz})$. Of the two, the average scala penetrometer reading is likely to provide a more reliable estimate of soil attenuation coefficient so its use is preferred.

2 Calculate the soil coefficient for the dominant frequency (f) of the construction activity of interest (column labelled 'Frequency' in table 4.1) as follows:

$$\alpha(f) = \frac{\alpha(5Hz)}{5} \times f \qquad \text{(Equation 8.1)}$$

3 Calculate the vibration level at distance from source of interest as follows:

$$V_2 = V_1 \left(\frac{10}{R}\right)^{0.5} e^{-\alpha(f)\times(R-10)} \qquad \text{(Equation 8.2)}$$

where: V_1 = the measured peak particle velocity at 10m distance from table 4.1

V_2 = the PPV at distance R (m) from source

In order to simplify the calculation process outlined above, a series of plots have been generated based on the attenuation and peak vibrations measured as part of this research project.

Using the main soil types from figures 6.1 and 6.2, we propose three soil attenuation categories as presented in table 8.1. These categories can be checked by using scala penetrometer tests, if available. Some judgement should be applied when using the scala maximum as the reading may just relate to a narrow band and not be representative of the soil material.

Table 8.1 Attenuation categories

Attenuation category	Typical soils	Scala reading (blows/50mm)	
		Maximum	Average
1	Weak or soft soils, including peat or organic soils and completely weathered soils	Below 5.5	Below 3
2	Competent soils, including sandy and silty clays, silt, weathered rock	5.5 to 10	3 to 6
3	Hard soils, including dense compacted sands, consolidated clay, sandstone	Above 10	Above 6
	Well sorted sands (ie uniform grain size), pumice sand		

With reference to the test results presented in section 6.2, the dune sand and pumice sand fall into category 3, which suggests in both cases a well compacted site.

Once the attenuation category has been established for the construction site of interest, figures 8.1 to 8.3 can be used to estimate the maximum vibrations expected from site preparation activities.

The vibration decay plots have been determined using the minimum attenuation value for the category, together with the source vibration and frequency for the particular item of equipment.

Superimposed on figures 8.1 to 8.3 are the suggested criteria for occupant disturbance and building damage. As can be seen, the separation distance from the source required to satisfy these criteria increases with increasing scala reading and decreasing vibration frequency.

Typically, for general site preparation works, a separation distance of between 12m and 20m is required for ground vibrations to be below the damage threshold of 5mm/s PPV and in excess of 50m for ground vibrations to be below the disturbance threshold of 0.5mm/s PPV.

Figure 8.1 Expected peak particle velocities – attenuation category 1

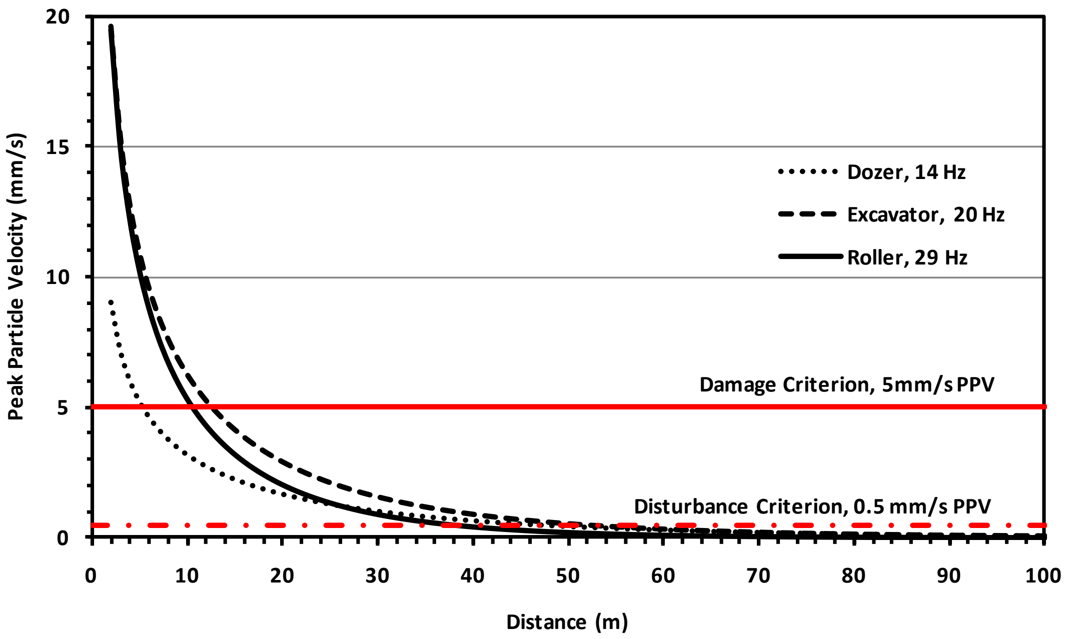


Figure 8.2 Expected peak particle velocities – attenuation category 2

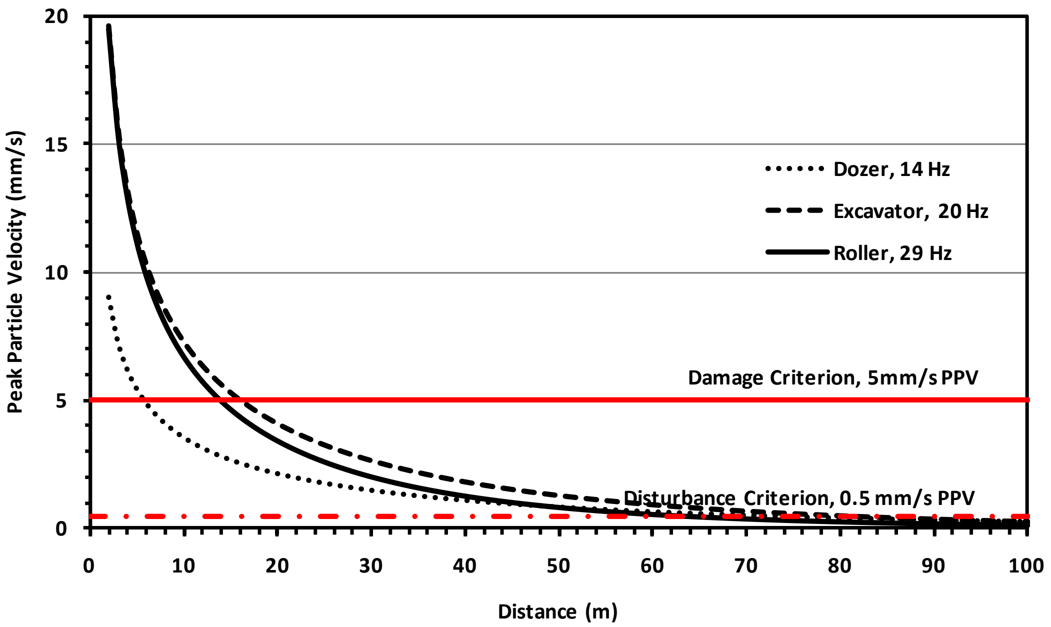
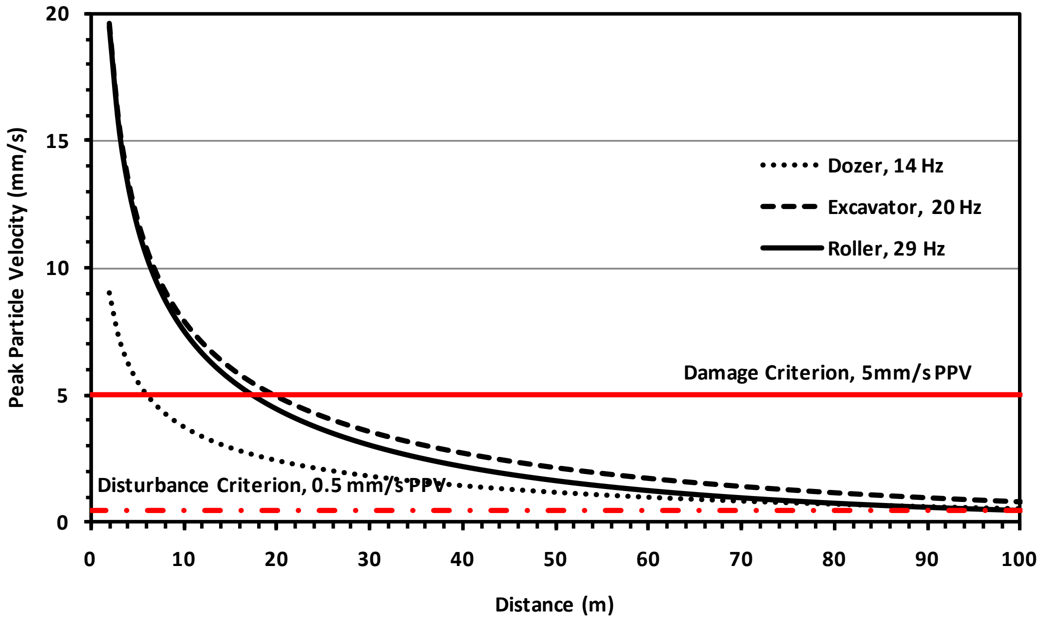


Figure 8.3 Expected peak particle velocities – attenuation category 3



8.1.2 British Standard BS 5228-2:2009

Ground vibration levels from vibratory compaction, percussive and vibratory piling, dynamic compaction, the vibration of stone columns and tunnel boring operations can be estimated using predictive equations given in table E.1 of BS 5228-2:2009. These equations have been reproduced in table 8.2 for ready reference. The original source of the equations is *TRL report 429* (Hiller and Crabb 2000).

Use of these equations enables a prediction to be made of the resultant PPVs at a specified distance from the source. For some construction activities the equations can provide an indication of the probability of the predicted vibration levels being exceeded.

With reference to the piling case studies presented in sections 5.1.1 and 5.1.2, the BS 5228-2:2009 predictive equations appear to be suitable for application in New Zealand, giving vibration levels that agree well with onsite measurements.

8.1.3 Falling weight deflectometer derived predictor curves

This method was discussed in chapter 7. It utilises the FWD to characterise the site. Measurements using the FWD and time histories are obtained to determine the PPV at each sensor location. The energy from the FWD is scaled with regards to the square root law and multiplied by the distance to each sensor to develop the scaled distance, SD. Both the PPV and SD for each sensor can then be plotted on a log-log graph to develop the predictor curve for the site.

To determine a particular mechanical equipment's impact on the surrounding area, the predictor curve and the energy of the equipment (whether real or assumed) can then be used to calculate the PPV at a particular distance.

This FWD-based method is recommended for estimating vibration levels from impact-related construction activities, such as compaction and piling. This is on the proviso that FWD time history data can be made available.

Table 8.2 Empirical predictors for groundborne vibrations arising from mechanised construction works (reproduced from table E.1, BS 5228-2:2009)

Operation	Prediction question	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%)	$1 \leqslant n_d \leqslant 2$ $0.4 \leqslant A \leqslant 1.72$ mm $2 \leqslant x \leqslant 110$ m
Vibratory compaction (start up and run down)	$v_{res} = k_t\sqrt{n_d}\left[\frac{A^{1.5}}{(x+L_d)^{1.3}}\right]$	$k_t = 65$ (50%) $k_t = 106$ (33.3%) $k_t = 177$ (5%)	$0.75 \leqslant L_d \leqslant 2.2$ m
Percussive piling	$v_{res} \leqslant k_p\left[\frac{\sqrt{W}}{r^{1.3}}\right]$	For piles at refusal: $k_p = 5$ For piles not at refusal: $1 \leqslant k_p \leqslant 3$, depending on soil type	$1 \leqslant L \leqslant 27$ m $1 \leqslant x \leqslant 111$ m (where $r^2 = L^2 + x^2$) $1.5 \leqslant W \leqslant 85$ kJ
Vibratory piling	$v_{res} = \frac{k_v}{x^{\delta}}$	$k_v = 60$ (50%) $k_v = 126$ (33.3%) $k_v = 266$ (5%)	$1 \leqslant x \leqslant 100$ m $1.2 \leqslant W \leqslant 10.7$ kJ $\delta = 1.3$ (all operations) $\delta = 1.2$ (start up and run down) $\delta = 1.4$ (steady state operation)
Dynamic compaction	$v_{res} \leqslant 0.037\left[\frac{\sqrt{W_h}}{x}\right]^{1.7}$		$5 \leqslant x \leqslant 100$ m $1.0 \leqslant W_h \leqslant 12$ 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%)	$8 \leqslant x \leqslant 100$ m
Tunnelling (groundborne vibration)	$v_{res} \leqslant \frac{180}{x^{1.3}}$		$10 \leqslant r \leqslant 100$ m
Tunnelling (groundborne noise)	$L_p = 127 - 54\log_{10} r$		$10 \leqslant r \leqslant 100$ m

A	maximum amplitude of drum vibration, in millimetres (mm)	v_{res}	resultant PPV, in millimetres per second ($mm \cdot s^{-1}$)
L	pile toe depth, in metres (m)	W	nominal hammer energy, in joules (J)
L_d	vibrating roller drum width, in metres (m)	W_c	energy per cycle, in kilojoules (kJ)
L_p	room octave band sound pressure level, in decibels (dB)	W_h	potential energy of a raised tamper, in joules (J)
n_d	number of vibrating drums	x	distance measured along the ground surface, in metres (m)
r	slope distance from the pile toe, in metres (m)		

8.2 Vibration management plans

Once a site has been assessed with regards to possible building damage or potential human impacts and a potential risk has been identified, control measures can then be identified. This requires consideration of equipment location and processes.

The Federal Transit Administration publication, *Transit noise and vibration impact assessment* (Hanson et al 2006) provides the following guidelines and information regarding management of vibrations:

1 Design consideration and layout:

   a Route heavily laden trucks away from residential streets, if possible. Select streets with fewest homes if no alternatives are available.

   b Operate earth-moving equipment on the construction lot as far away from vibration-sensitive sites as possible.

2 Sequence of operations:

   a Phase demolition, earth-moving and ground-impacting operations so they do not occur in the same period. Unlike noise, the total vibration level produced could be significantly less when each vibration source operates separately.

   b Avoid night-time activities. People are more aware of vibration in their homes during the night-time hours.

3 Alternative construction methods:

   a Avoid impact pile-driving where possible in vibration-sensitive areas. Drilled piles or the use of a sonic vibratory pile driver causes lower vibration levels where the geological conditions permit their use. However, refer to the cautionary note below.

   b Select demolition methods not involving impact, where possible. For example, sawing bridge decks into sections that can be loaded onto trucks results in lower vibration levels than impact demolition by pavement breakers, and milling generates lower vibration levels than excavation using clam shell or chisel drops.

   c Avoid vibratory rollers and packers near sensitive areas.

According to Hanson et al (2006), pile-driving is one of the greatest sources of vibration produced by construction equipment. Sonic pile drivers, which operate by vibrating the piles into the earth at a fixed frequency, may provide a significant reduction in vibration magnitude. They can, however, cause some additional vibration effects that may restrict their use. For example, sustained vibration at a fixed frequency may be more noticeable to nearby residents, and the steady-state excitation of the ground may result in an increase in the resonant response of building components. This increase in resonant response may be unacceptable in the proximity of vibration sensitive structures and land use such as hospitals, listed buildings, precision laboratories and communication towers. By comparison, impact pile-drivers produce large vibrations, but the relatively long duration between impacts allows the resonance to decay.

Furthermore, in situations where there is insufficient information available during the preliminary engineering phase to clearly define the required vibration management measures, it is appropriate to describe and commit to a vibration management plan that will be developed and implemented at a later stage. The plan's objective should be to minimise damage due to construction vibration by all reasonable and feasible means possible, and should include a procedure for determining vibration limits and thresholds for each potentially affected structure, based on an assessment of their ability to withstand vibration and displacement. It should also include a plan for monitoring the construction vibration, and assessing compliance with the established vibration thresholds (Hanson et al 2006).

9 Conclusions and recommendations

The principal conclusions and recommendations arising from this research programme, which was concerned with the reliable prediction of ground vibration levels from road construction activity over a range of distances from the vibration source, are presented below.

9.1 Conclusions

9.1.1 Assessment of effects of ground vibrations

Methods for predicting ground vibrations, unlike physical measurements, cannot characterise the vibrations in terms of their individual frequency components (ie frequency spectrum). Therefore, their output is limited to the amplitude of the vibration, which typically is in terms of PPV. From a consideration of various vibration-related standards, the following two criteria are suggested for preliminary assessment of the predicted ground vibrations:

- 0.5mm/s PPV for disturbance of building occupants;
- 5mm/s PPV for building damage.

However, these criteria do not apply to soils susceptible to vibratory densification/liquefaction. In general, these soils comprise uniform grain size (ie low cohesion) silts below the water table and have a very low scala count, ie about two blows per 50mm. For such cases, specialist geotechnical advice should be sought on the vibration levels that could cause consolidation or densification of the soil, which would result in differential settlement and consequent building damage.

9.1.2 Measured vibration levels

- Ground vibrations measured during site preparation activities on representative New Zealand soil types were found to span a wide range of magnitude from 0.4mm/s PPV to 11mm/s PPV when standardised for a distance of 10m from the vibration source.
- No significant correlation between weight of the equipment and the resulting ground vibration was observed, with the heaviest piece of machinery, a 33 tonne static roller, inducing one of the lowest measured ground vibration levels (0.7mm/s PPV at a distance of 10m).
- There can be a significant difference in the level of ground vibration generated depending on the make and model of the equipment.
- Contrary to expectation, softer soils did not always produce the larger magnitude vibrations.
- Pile driving and vibratory compaction generated the most severe vibrations.
- Piles driven to refusal (ie point at which the pile cannot be driven deeper) generated larger magnitude ground vibrations than piles just driven through the soil.
- Vibratory piling, which is the vibrating of piles into the ground at fixed frequency, provided a significant reduction in vibration magnitude (quarter to half) over impact piling, particularly in easily mobilised soils, such as well sorted sand.
- Measured vibration levels from impulsive/percussive pile driving and vibratory pile driving agreed well with estimated vibration levels using empirical formulae given in appendix E of BS5228-2:2009.

9.1.3 Soil attenuation

Soil attenuation characteristics determine the rate at which ground borne vibrations decay with distance from source. It was observed that vibrations of higher frequency decayed faster than those of lower frequency and so any modelling of soil attenuation should take into account the dominant frequency of the vibration source.

The larger the value of the soil attenuation coefficient, the more rapidly the ground-borne vibrations decayed with distance from the vibration source.

Soil attenuation properties derived for main soil types found in New Zealand are as summarised in the table below:

Description of soil	**Frequency independent soil property, ρ (s/m)**	**Attenuation coefficient, α, at 5 Hz (m^{-1})**
Silty sand	2.04×10^{-4} – 3.18×10^{-4}	0.003 – 0.005
Sandstone	1.47×10^{-4} – 6.12×10^{-4}	0.002 – 0.01
Pumice fill	8.22×10^{-4} – 1.23×10^{-3}	0.013 – 0.019
Peat	8.30×10^{-4} – 1.94×10^{-3}	0.013 – 0.03

The frequency independent material property of soil, ρ (s/m), is related to the frequency dependent attenuation coefficient, α (m^{-1}) by the following relation:

$$\alpha = \rho\pi f \text{ where } f = \text{frequency of source vibration } (Hz)$$

Both average and maximum scala penetrometer readings were shown to be very good predictors of soil attenuation, even for volcanic soils, with a coefficient of determination (r^2) of about 0.9 being achieved. However, because the maximum scala reading may just relate to a narrow band and may not be representative of the soil material, the equation below should be used in obtaining estimates of site-specific soil attenuation coefficient:

$$\alpha(5Hz) = 0.0364e^{-0.411 SCALA_{avg}}$$

where: α(5Hz) = soil attenuation coefficient (m^{-1}) for a frequency of 5Hz

$SCALA_{avg}$ = average scala reading (blows/50mm)

9.1.4 Use of falling weight deflectometer data

Displacement time histories obtained during FWD measurements can be used to generate site-specific predictor curves for estimating vibration levels from impact-related construction activities such as compaction and piling. This method makes use of a square-root scaling law of ground motion and does not require detailed knowledge of the layering of the pavement structure or the geology of the surrounding site.

9.1.5 Prediction and management

Three methods are proposed for quantifying vibration impacts from construction activity as follows:

1 Previously measured vibration source levels in combination with site scala penetrometers results. This method is suggested for equipment associated with site preparation. The vibration level at distance R_2 from the source of interest is calculated from:

$$V_2 = V_1 \left(\frac{R_1}{R_2}\right)^{0.5} e^{-\alpha(f)\times(R_2-R_1)}$$

where: V_1 = the measured peak particle velocity (mm/s) at distance R_1 (m)

V_2 = the peak particle velocity (mm/s) at distance R_2 (m) from source

$\alpha(f)$ = soil coefficient for the dominant frequency f (Hz)

2 Empirical formulae given in BS 5228-2:2009. This method is suggested for vibratory compaction, impact and vibratory piling, dynamic compaction, the vibration of stone columns and tunnel boring operations.

3 FWD derived predictor curves. This method is suggested for impact related construction activities, such as compaction and piling, wherever FWD data y is already available.

On the basis of the field measurements, the following indicative separation distances are required to ensure ground vibrations generated from construction activity are below the damage threshold of 5mm/s PPV:

- >20m for general site preparation works
- >30m for vibratory piling
- >70m for impact/percussive piling.

9.2 Recommendations

- The relationship between scala penetrometer readings and soil attenuation needs to be validated over a wider range of New Zealand soil types before it can be applied with confidence.
- Similarly, the FWD method for generating site-specific predictor curves has not been validated under New Zealand conditions. Therefore, whenever the opportunity arises, the method should be trialled.
- The NZTA already has a comprehensive coverage of FWD measurements over the state highway network. These measurements can be accessed through the FWD table in the road assessment and management (RAMM) database. In order to permit ready derivation of project-specific ground motion predictor curves from the FWD time histories, investigations should be undertaken to establish if peak velocities at each of the sensor locations can be retrospectively generated for the FWD data already in RAMM and for future FWD surveys if the peak velocities could be recorded in addition to the peak displacements.
- Over time, it would be beneficial to build up a vibration-risk map of New Zealand that would have as its basis measured soil attenuation coefficients. Therefore, when conducting ground vibration measurements for assessment purposes, it would be desirable to describe soils in a consistent manner to reduce any subjective nature and variability in the reporting. The method of soil and rock description contained in the NZ Geotechnical Society's (2005) guideline for the field classification and description of soil and rock for engineering purposes appears to be the most appropriate for field use and so is recommended. This guideline can be downloaded from the following link, which was last accessed in March 2012, www.nzgs.org/wp-content/uploads/soil_and_rock_field_guide.pdf

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Appendix A: Photographs of equipment

MacKay's Crossing

Equipment	Dynapac CC422HF
Weight	10.4 tonne
Soil type	Recompacted weathered greywacke

Equipment	Caterpillar D4H
Weight	7.9 tonne
Soil type	Peat

Equipment	Bomag BW177D-3
Weight	7.2 tonne
Soil type	Peat

Meeanee Interchange

Equipment	Hamm 3410
Weight	11.015 tonne
Soil type	Marine silt

Equipment	Komatsu D 65 E
Weight	20.280 tonne
Soil type	Marine silt

Rotorua

Equipment	Komatsu PC60
Weight	6 tonne
Soil type	Ash

Equipment	Sakai SW 500
Weight	4.0 tonne
Soil type	Recompacted pumice

Equipment	Sumitomo SH120
Weight	12.6 tonne
Soil type	Recompacted pumice

Equipment	Komatsu D31P
Weight	7.3 tonne
Soil type	Recompacted pumice

Rotorua continued

Equipment	Truck and trailer unit (Make of truck Hino)
Weight	44 tonne
Soil type	Silt

Mt Roskill

Equipment	Caterpillar 815B
Weight	20.8 tonne
Soil type	Recompacted ash

Equipment	Caterpillar CS663E
Weight	17.1 tonne
Soil type	Recompacted ash

Equipment	Caterpillar 825C
Weight	32.7 tonne
Soil type	Recompacted ash

Greenhithe

Equipment	Caterpillar CB544
Weight	10.7 tonne
Soil type	Recompacted ash

Foxton

Equipment	Sakai SW70C
Weight	7.1 tonne
Soil type	Dune sand

Alpurt

Equipment	Caterpillar CP 663E
Weight	16.8 tonne
Soil type	Waitemata sandstone

Equipment	Caterpillar 825C
Weight	32.7 tonne
Soil type	Waitemata sandstone

Equipment	Caterpillar D6R
Weight	20.6 tonne
Soil type	Waitemata sandstone

Equipment	Dynapac CA602
Weight	18.6 tonne
Soil type	Waitemata sandstone

Taupo

Equipment	Hamm 3414
Weight	14.3 tonne
Soil type	Pumice sand

Equipment	Volvo G726B
Weight	16 tonne
Soil type	Pumice sand

Equipment	Wirtgen WR2000
Weight	22 tonne
Soil type	Pumice sand

Christchurch

Equipment	Sakai SW70C
Weight	7 tonne
Soil type	Silt

Appendix B: Glossary of key terms

Cohesionless soils	Any free-running type of soil, such as sand or gravel, whose strength depends on friction between particles.
Compression (P-) waves	This is the fastest kind of seismic wave, and, consequently, the first to 'arrive' at a seismic station. The P-wave can move through solid rock and fluids. It pushes and pulls the rock it moves through just like sound waves push and pull the air.
Continuous vibration	Vibrations that continue uninterrupted for a defined period (usually throughout day-time and/or night-time). Examples include steady road traffic or continuous construction activity such as tunnel boring.
Impulsive vibration	Vibrations that build up rapidly to a peak followed by a damped decay but may or may not involve several cycles (depending on frequency and damping). It can also consist of a sudden application of several cycles at approximately the same amplitude, providing that the duration is short, typically less than two seconds. It relates to infrequent activities that create up to three distinct vibration events in an assessment period. An example would be occasional loading and unloading.
Intermittent vibration	Interrupted periods of continuous (eg a drill) or repeated periods of impulsive vibration (eg a pile driver), or continuous vibration that varies significantly in magnitude. It may originate from impulsive sources (eg pile drivers) or repetitive sources (eg pavement breakers) or sources that operate intermittently, but which would produce continuous vibration if operated continuously (eg heavy traffic passing by).
Rayleigh (R-) waves	An R-wave rolls along the ground just like a wave rolls across a lake or an ocean. Because it rolls, it moves the ground up and down, and side-to-side in the same direction that the wave is moving. Most of the shaking felt from an earthquake is due to the R-wave, which can be much larger than the other waves.
Scala penetrometer	Also known as dynamic cone penetrometer. It comprises a case hardened cone driven into the soil by successive blows of a 9.1kg hammer dropping 510mm vertically onto a steel anvil. A record of depth of penetration for each blow of the hammer is kept by the operator as the cone is driven through the soil strata under investigation.
Shear (S-) waves	S-waves move rock particles up and down, or side-to-side--perpendicular to the direction that the wave is travelling in (the direction of wave propagation). An S-wave is slower than a P-wave and can only move through solid rock, not through any liquid medium.