2008:06

TECHNICAL REPORT

Train-induced Vibrations in Tunnels – A Review

Andreas Eitzenberger

Luleå University of Technology
Department of Civil, Mining and Environmental Engineering
Division of Mining and Geotechnical Engineering

2008:06 • ISSN: 1402 - 1536 • ISRN: LTU - TR - - 08/06 - - SE

TECHNICAL REPORT

Train-induced Vibrations in Tunnels – A Review

by

Andreas Eitzenberger

Division of Mining and Geotechnical Engineering
Luleå University of Technology
SE-971 87 Luleå
SWEDEN

PREFACE

This literature review is the first part of a research project financed by Banverket. The aim of the research project is to increase the understanding on how train-induced vibrations are affected by the properties of the rock through which it propagates. In this report, the results of a literature review covering several different areas, such as: wave propagation in geomaterials, methods for analysis of vibrations, national and international regulations, and countermeasures for train-induced vibrations (just to name a few), are presented.

I would like to thank those that in various ways have supported and helped me along the way. They are: Erling Nordlund at the division of Geotechnology, Luleå University of Technology, Olle Olofsson, Peter Lundman, and Alexander Smekal, all at Banverket, and Catrin Edelbro at the division of Geotechnology, Luleå University of Technology.

Luleå, April 2008,

Andreas Eitzenberger

SUMMARY

Banverket is expecting that the number of railway tunnels in densely populated areas will increase over the next 20 years due to the lack of available space on the ground surface. Together with the increased awareness of the residents the need for good prediction of vibration and noise levels in dwellings along the planned tunnels is evident. Consequently, a study of the propagation of vibrations through rock and soil generated by trains operating in tunnels is required in order to make more reliable prognoses.

This report constitutes the first stage within a research project aimed at increasing the understanding about ground-borne noise and ground-borne vibrations generated by trains moving in tunnels constructed in rock. In this report, the propagation of vibration through a rock mass is reviewed. The emphasis has been on wave propagation in hard rock, but soil has also been included. Areas, such as the generation of vibration at the train-rail interaction, the response of buildings and humans, national and international recommended noise and vibrations levels, measurement of noise and vibrations, and possible countermeasures are briefly reviewed as well. Finally, suggestions for the continued research within this field are presented.

The propagation of waves is influenced by attenuation along the propagation path. The attenuation can either be through geometric spreading, energy loss within the material, or reflection and refraction at boundaries. In a rock mass, where heterogeneities of various scales are present, the attenuation of (train-induced) waves through the ground therefore mainly depends on discontinuities, e.g. joints, faults, cracks, crushed zones, dykes, and boundaries between different rock types or soil. Also the topography – along as well as intersecting tunnels – influences the wave propagation in form of local amplification. An increased amount of joints, faults and boundaries increases the attenuation of the waves.

The rock mass is in most cases inhomogeneous due to all heterogeneities present. Despite this fact, the rock mass and soil is always treated as an isotropic, homogeneous material when analyzed with regard to ground-borne noise and ground-borne vibrations. This concerns both numerical and empirical methods. Thus, there is a lack of knowledge regarding the influence of various heterogeneities on the propagation of waves, and thereby vibrations, in non-isotropic ground conditions (e.g. a rock mass) at low frequencies.

Future research regarding train-induced vibration should focus on conceptual models used to determine the propagation of low-frequency waves in a rock mass containing various amount of heterogeneities (from isotropic to highly inhomogeneous). Once the behaviour of waves in

an inhomogeneous rock mass has been established, conceptual models should be used together with measurements from a few well documented cases. From the results of the analysis, guidelines for analysis of railway tunnels with regard to ground-borne noise and ground-borne vibrations should be established.

Keywords: Ground-borne vibration, ground-borne noise, rock mass, tunnel, train, soil.

CONTENT

1 INTRODUCTION

1.1 Background

In densely populated areas there is a lack of available surface space. This causes problems when the capacity on roads and railways has to be increased. A common solution is to build the new roads and railways underground. Some example of current projects in Sweden is Södra länken in Stockholm, where parts of the new traffic route was constructed underground, and Citybanan in Stockholm, which will increase the allowed train capacity on the south bound rout. Another example is Citytunneln in Malmö which has the purpose of changing the train station from a terminal station (terminus) to a non-terminus station (trains do not have to reverse).

When trains move along the railway (along open track as well as in tunnels) noise and vibrations are generated. The noise and vibrations will radiate away from the railway, and they may cause disturbance to the nearby residents. In many cases measures are taken in order to reduce the noise and vibrations. For open track railways extensive knowledge and numerous solutions are available for dealing with noise and vibration related problems. Furthermore, the generation of vibrations (train-rail interaction) as well as the behaviour of the receiver (buildings and humans) is rather well understood today. However, a better understanding of the propagation of vibration through the ground (rock and soil) generated by train in tunnels is needed.

Banverket is expecting an increase of railroad tunnels in densely populated areas over the next 20 years. Due to the increased awareness of the residents, with respect to their living environment, the need for reliable prediction of vibration levels along with solutions to reduce the induced vibrations is evident. In order to be able to achieve this one has to understand the propagation of waves from the source (train) to the surrounding buildings and their inhabitants. Hence, a study of the propagation of vibrations through rock and soil is necessary.

1.2 Objective and limitations

The aim of this study has been to review the current state of the art concerning the propagation of train-induced vibrations through the ground. Vibrations and their propagation is well known for homogenous materials, but not as well understood for in-homogenous materials like rock and soil. Several branches, such as; acoustics (ground vibrations and noise), rock engineering (blasting), seismology (earthquakes), geophysics (investigations methods) and structural dynamics (building vibrations), has therefore been reviewed. Methods, empirical as well as numerical, used to determine the propagation of (train-induced) vibration through the ground, have been reviewed. Also the influence of various heterogeneities on the propagation of waves through the ground has been reviewed.

The emphasis has been on the propagation of vibration through the ground, foremost rock, but also soil. The vibrations have to be induced by trains moving in tunnels, or by forces similar to those induced by train. The generation of vibrations (wheel-rail interaction) as well as the response of buildings and humans are believed to be well understood and these areas are therefore only briefly reviewed.

1.3 Outline of report

Chapter 1 contains background, aim and limitations of the study, as well as an outline of the report. Also a general introduction to vibration and noise is given within the chapter.

In chapter 2 the generation, propagation and receiving of train-induced vibrations are reviewed as well as measures to reduce vibrations.

In chapter 3 wave propagation through rock and soil is described together with an explanation of how they are affected by various changes along the propagation path. Empirical as well as numerical methods used to study train-induced vibration and its effect on the surrounding are briefly reviewed.

Recommended levels for noise and vibrations from several national and international codes and regulations are reviewed in chapter 4. Also standards specified for train in tunnels are briefly reviewed.

The basics of noise and vibration measurements are briefly explained in chapter 5. Also the guidelines from international standards for measurements of ground-borne noise and ground-borne vibrations are included within the chapter.

A number of cases are presented in chapter 6. For each case, either the measurements and/or the analysis performed to determine the ground-borne noise or ground-borne vibrations around the tunnels, is briefly described.

A discussion on the subject of train-induced vibration and its propagation from train to building are presented in chapter 7. The chapter also contains the conclusions and suggestions for further research.

1.4 Introduction to noise and vibrations

Trains moving along open tracks as well as underground generate vibrations that will propagate away from the source. Regardless of path taken, since noise can propagate through air, water and solid materials, the waves will end up as noise or vibration in buildings along the railway. Theories regarding the wave movements are therefore bases on laws of physics that are very similar. Within the following subsections a brief introduction to some basics regarding noise and vibrations is given.

Time and frequency domain

The simplest way to illustrate sound or vibrations is as a harmonic motion. This can either be done in the time domain, where the amplitude is represented as a sinusoidal function of time, or in the frequency domain, where the amplitude is represented as a function of frequency (Figure 1.1a). A harmonic sound consists of one frequency, f, which would be heard as a continuous pure tone. Sometimes two harmonic motions with different frequencies will coincide and a superimposed motion will be created (Figure 1.1b). A more complex signal would be the square motion consisting of several frequencies (Figure 1.1c). These three motions can be determined as periodic. Noise in general cannot be described as a simple harmonic motion, and is therefore non-periodic. To present noise in the frequency plane, the signal would have to be divided into several small contiguous frequency bands where an average of each band would be determined (Figure 1.1d). This would result in a spectrum and is a common way to illustrate the noise and vibration induced by trains. It is only possible to measure the amplitude of the noise and vibration as a function of time, i.e. in the time domain. The amplitude as a function of frequency can be acquired from Fourier transformation.

Octave bands

Dividing the frequencies of a broad band sound or vibration into several small contiguous bands will make the characterization easier. For hearable sound (20 Hz to 20 000 Hz) this usually results in 8 to 11 octave bands, while for vibrations, a few more bands are added in the low frequency range. The octave bands and their corresponding centre frequency are shown in Table 1.1 (the third column). Octave band is satisfactory for community noise control. When higher accuracy is needed smaller bands are required. Usually each octave band is divided into three bands, resulting in one-third octave bands (rightmost column in Table 1.1). If even higher accuracy is required, e.g. for product design and when trouble shooting industrial machines, band widths down to 2 Hz are sometimes used.

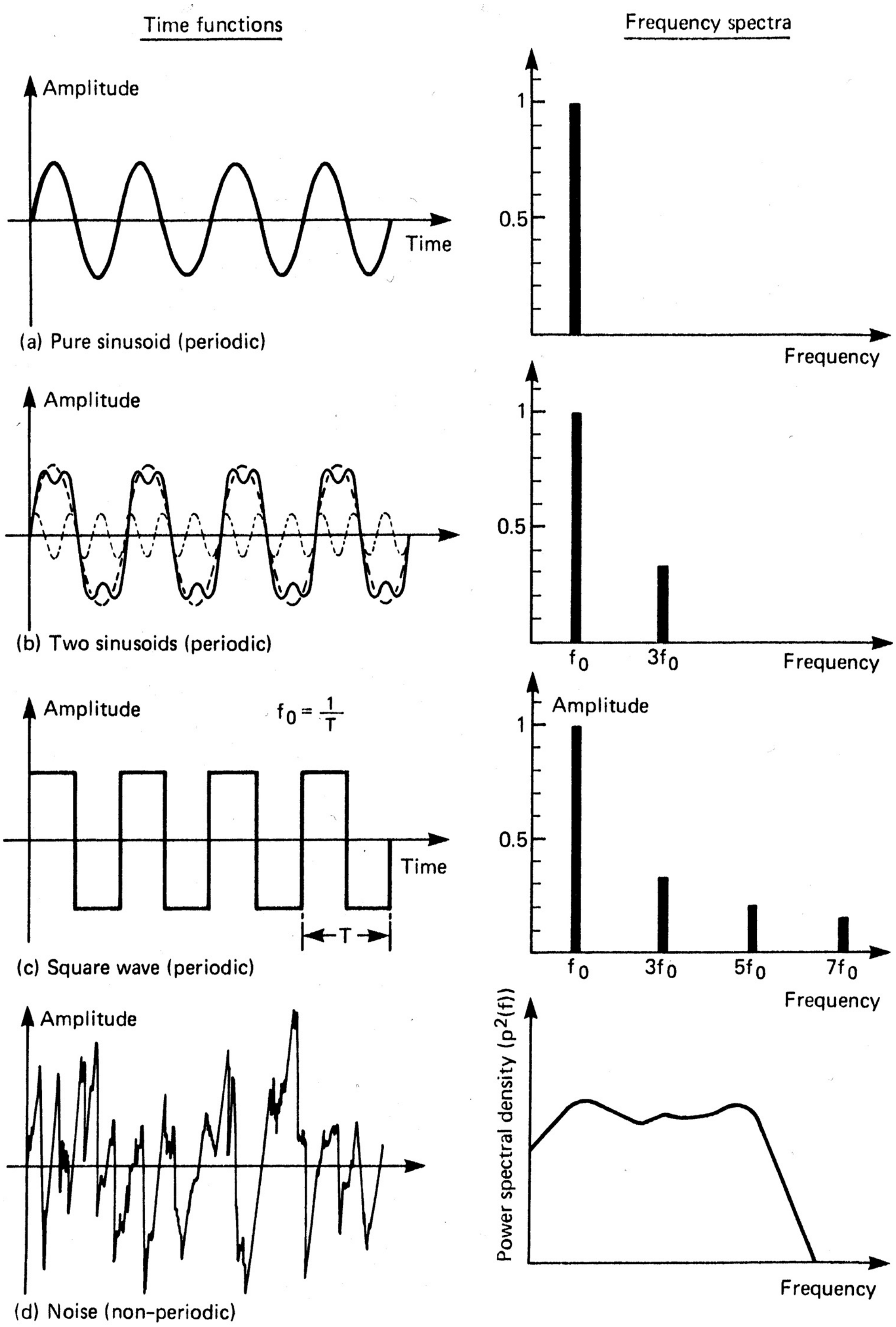


Figure 1.1. Waveforms and their spectra for (a) single sinusoidal motion, (b) two sinusoidal motions, (c) square wave motion, and (d) noise (from Ford, 1987).

Table 1.1. Standardized centre, upper and lower limiting frequencies for octave (bold) and one-third octave bands within the frequency range 1 to 315 Hz (Bodén et al., 2001; Ford, 1987).

Band Nr.	Center Frequency [Hz]	Octave frequency range (band) [Hz]	One-third octave frequency range [Hz]
1	1.25	1.12-1.41	-
2	1.6		1.41-1.78
3	**2**	**1.41-2.82**	1.78-2.24
4	2.5		2.24-2.82
5	3.15		2.82-3.55
6	**4**	**2.82-5.62**	3.55-4.47
7	5		4.47-5.62
8	6.3		5.62-7.08
9	**8**	**5.62-11.2**	7.08-8.91
10	10		8.91-11.2
11	12.5		11.2-14.1
12	**16**	**11.2-22.4**	14.1-17.8
13	20		17.8-22.4
14	25		22.4-28.2
15	**31.5**	**22.4-44.7**	28.2-35.5
16	40		35.5-44.7
17	50		44.7-56.2
18	**63**	**44.7-89.1**	56.2-70.8
19	80		70.8-89.1
20	100		89.1-112
21	**125**	**89.1-178**	112-141
22	160		141-178
23	200		178-224
24	**250**	**178-355**	224-282
25	315		282-355

1.4.1 Noise

The human ear can register pressure variations as small as 20 µPa (lowest value) and as high as 20 Pa, which is the pain threshold. Due to the large range Alexander Graham Bell introduced a logarithmic scale to describe the sound pressure, were the measured value is divided with a reference value. The scale is divided into levels and is measured in decibel (dB). Sometime the measures Bel (dB = 1/10 Bel) can be encountered.

Sound power level (SWL) is a measure of the acoustical power or sound effect that is radiated from the source to the surrounding and is defined as:

$$L_W = 10 \cdot \log \frac{W}{W_0}, \qquad [\text{dB}] \tag{1.4}$$

where W is the sound power and W_0 is a reference power (10^{-12} W). The *Sound intensity level* (SIL) can be determined in a similar way

$$L_I = 10 \cdot \log \frac{I}{I_0}, \qquad [\text{dB}] \tag{1.5}$$

where I is the sound power and I_0 is a reference intensity (10^{-12} W/m^2). The *Sound pressure level* (SPL) is a measure of the strength of the sound (noise) and is defined as:

$$L_p = 20 \cdot \log \frac{p}{p_0}, \qquad [\text{dB}] \tag{1.6}$$

where p is the effective sound pressure and p_0 is a reference pressure, which is 20 µPa ($2 \cdot 10^{-5}$ Pa) and barely hearable by humans. A sound pressure of 20 µPa corresponds to a sound pressure level of 0 dB while 20 Pa correspond to the sound pressure level of 120 dB. The SPL is usually the quantity used when dealing with noise.

Loudness

The ear of a young male will respond to sound in the frequency range of 20 to 16 000 Hz, while for children and women the ear can respond to sound with frequencies up to 20 000 Hz. Below 20 Hz is the infra sound and above 20 000 Hz is the ultra sound. The ear is most sensitive in the frequency range from 2 000 to 5 000 Hz. For a frequency of 1 000 Hz this corresponds to a movement of air molecules in the range of 1.0 nm. Speech is within the frequency range of 500 to 2 000 Hz.

The human ear will apprehend noise differently for various frequencies since the ear is not equally sensitive for all frequencies. Thus, two noises with different frequency content but the same sound pressure levels will be recognized to have different loudness. Lower frequency noise has a lower loudness than high frequency noise. This was first observed by Fletcher and Munson in 1933. They conducted tests aimed at determining the relationship between frequency and loudness. Test persons were asked to adjust the sound level of the test tone so it would match the reference tone. This experiment resulted in the Fletcher-Munson equal loudness contours and is measured in phons, as shown in Figure 1.2. The lowest contour (dashed line) represents the threshold for hearing. This threshold may vary as much as ±10 dB between individuals (with normal hearing). Hence, loudness is something that is highly individual.

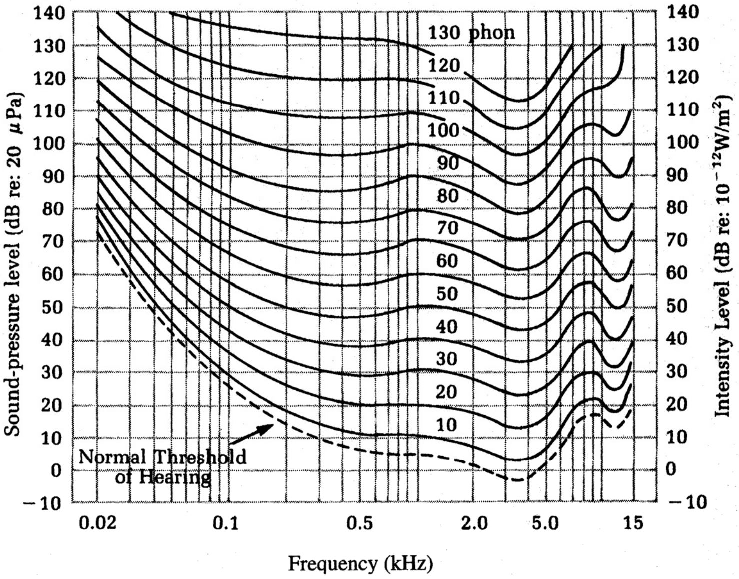


Figure 1.2. Equal loudness contours for humans hearing (from Davis and Cornwell, 1998).

A, B, C, and D-weighting

As mentioned above, noise is apprehended differently between individuals and is influenced by the frequency content of the noise. High frequency noise is, as seen in Figure 1.2, considered as more annoying at a lower threshold than low-frequency noise for the same sound pressure level. Due to this non-linear behaviour of the human ear, noise is measured and weighted in four classes; A, B, C, and D. These classes filter the noise levels differently for different frequencies, as shown in Table 1.2. Weighting class A filters the low frequencies heavily and is based on the 40 phon equal loudness contour, while class B only moderately filters the low frequencies and is based on the 70 phon equal loudness contour. Weighting class C hardly filters any frequency at all since it is based on the 90 phon equal loudness contour. There is also weighting class D which only is used when measuring noise from air planes. A weighting is the most commonly, although C-weighting is sometimes used for impulse noise. Sound pressure levels that are weighted are called sound levels. In order to show what weighting class that is used, sound levels are presented (for weighting class A) as dB(A), dBA, dBa, or L_A.

Table 1.2. Correction values for A-, B-, C- and D-weighting frequencies between 20 and 250 Hz (Bodén et al., 2001; Ford, 1987).

Frequency [Hz]	A-weighting [dB]	B-weighting [dB]	C-weighting [dB]	D-weighting [dB]
20	-50.5	-24.2	-6.2	-20.6
25	-44.7	-20.4	-4.4	-18.7
31.5	**-39.4**	**-17.1**	**-3.0**	**-16.7**
40	-34.6	-14.2	-2.0	-14.7
50	-30.2	-11.6	-1.3	-12.8
63	**-26.2**	**-9.3**	**-0.8**	**-10.9**
80	-22.5	-7.4	-0.5	-9.0
100	-19.1	-5.6	-0.3	-7.2
125	**-16.1**	**-4.2**	**-0.2**	**-5.5**
160	-13.4	-3.0	-0.1	-4.0
200	-10.9	-2.0	0	-2.6
250	**-8.6**	**-1.3**	**0**	**-1.6**

Equivalent sound pressure level

Noise can be measured over a defined time period. Over the time period the noise level will fluctuate. The equivalent noise level is a measure of the average noise pressure level over a given time period. For the same time period, it is believed that constant noise level expends the same amount of energy as the fluctuating noise level. Over a specified time interval the equivalent sound pressure level, L_{eq}, is determined using

$$L_{eq} = 10 \log \frac{1}{\tau} \int_0^{\tau} 10^{L/10} dt \qquad (1.7)$$

where τ is the time over which L_{eq} is determined, and L is the time varying noise level in dB(A). Due to the definition of the equivalent noise level, short strong noise will have a large effect on the equivalent noise level. For example, a 15 min exposure of 100 dB(A) is equal to 8 hours exposure of a noise level of 85 dB(A) (Bodén *et al.*, 2001).

Noise can be described as steady-state (or continuous), intermittent, or impulse (or impact), depending on the variations in levels of the measured time. Noise that vary less then 5 dB over the measurement time is regarded as continuous. Intermittent noise is continuous noise that is present for more than one second and is then interpreted for more than one second. Impulse noise has a duration that is less than one second and contains a sound pressure change of 40 dB within less than 0.5 second.

1.4.2 Vibrations

The magnitude of a vibration can be measured in several ways. The three most common ways are displacement (mm), velocity (mm/s), and acceleration (mm/s^2). It is easy to convert between the different unities as long as it is assumed that the vibration can be expressed as sinusoidal (harmonic) motion. The relation is

$$a = v(2f\pi) = d(4f\pi) \tag{1.8}$$

where a is the acceleration of the vibration, v is the velocity of the vibration, d is the displacement of the vibration, and f is the frequency of the vibration studied. Recommended vibration levels are usually expressed in terms of velocity or acceleration. Moreover, vibrations can also be expressed in levels (in the same way as sound pressure). Converting between acceleration and level (dB) can be done through the relation

$$L_a = 20\log\frac{a}{10^{-6}} \qquad \text{[dB]} \tag{1.9}$$

where L_a is the acceleration level (dB), a is the vibration acceleration, and 10^{-6} m/s^2 is the reference level for the acceleration. Vibration velocity can be converted to vibration levels in the same way by

$$L_v = 20\log\frac{v}{10^{-9}} \qquad \text{[dB]} \tag{1.10}$$

where L_v is the vibration level (dB), v is the vibration velocity, and 10^{-9} m/s^2 is the reference level for the velocity.

The attenuating effect of materials along with the vibration levels generated by the train is in the majority of references expressed in dB while the recommended values in standards are in velocity or acceleration.

2 TRAIN-INDUCED VIBRATIONS

2.1 Introduction

Trains moving along an underground railway will excite the rail and the underlying track structure. These vibrations will radiate into the surrounding ground, i.e. rock and soil, as *ground-borne vibrations*. The propagating path of the vibration will depend on the composition of the ground, as seen in Figure 2.1. The composition of the ground will also affect the amplitude and velocity of the propagating vibration. Once the vibrations reach a building they can either be felt by residents as whole body vibrations, or heard as low-frequency rumble, i.e. *ground-borne noise*. Vibrations can also cause structural damage or disturb sensitive equipment.

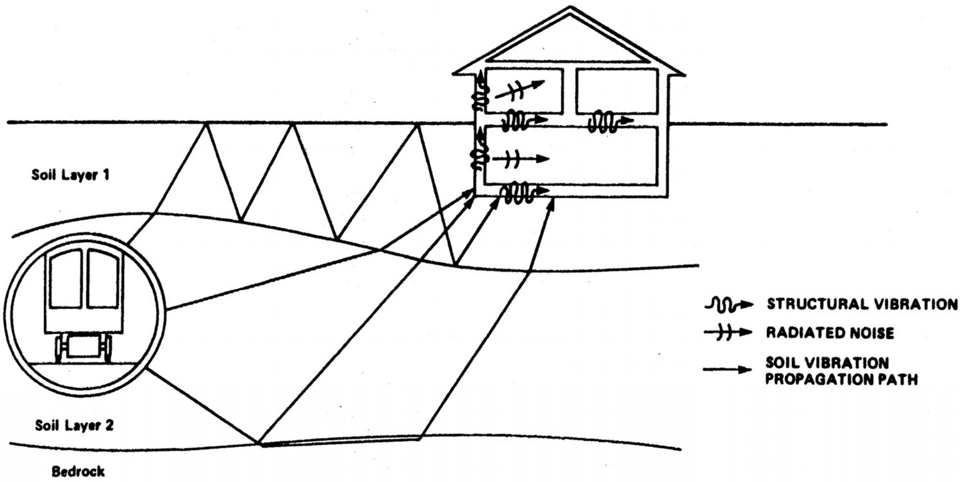


Figure 2.1. Different propagation paths for train-induced vibrations (Remington et al., 1987).

The transmission of vibrations from the moving train into the surrounding ground and onward to nearby buildings is complex and depends on several factors. It is therefore common to divide the generation and propagation of train-induced vibrations into three parts or stages; (i) the source, (ii) the propagation path, and (iii) the receiver (Figure 2.2). Melke (1988) defined the source to consist of the train and track, the path to be the tunnel (construction) and ground (transmission path), and the receiver to be the building. Since building has residents they also have to be treated as receivers. Knowledge about one stage can be used as input to a consecutive stage.

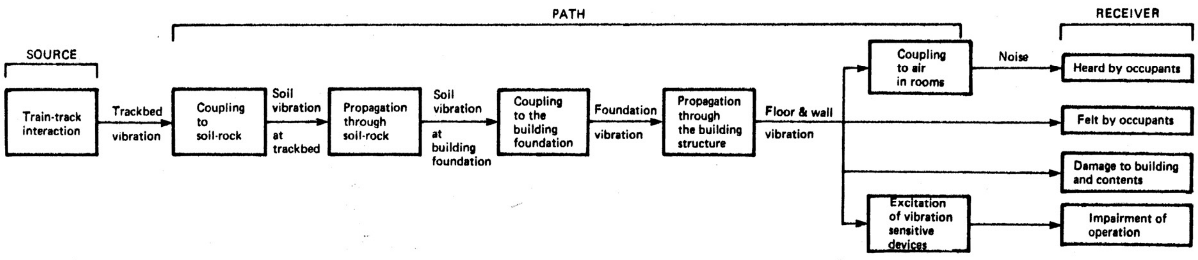


Figure 2.2. Block diagram illustrating the different stages when studying train-induced vibrations (Remington et al., 1987)

During the propagation from source to receiver the waves will be attenuated in various ways, but amplification may also occur. The attenuation (and amplification) is therefore of great interest when dealing with train-induced vibrations and its effects on the surrounding (humans or buildings). Consequently, the three stages (source, path and receiver) are briefly reviewed within the following sections. Common measures to reduce train-induced vibration are reviewed as well.

2.2 Vibration source

The source to train-induced vibrations is the movement of the train along the track and the interaction occurring between wheel, rail, and track structure. Trains standing still on a railway generate a force due to the weight that is transmitted from wheel to rail and redistributed by the rail, sleeper, ballast and ground. This load can be defined as static. When the train moves this force will move along with the train. The load will fluctuate due to differences at various parts of the train-track structure system, such as (i) irregularities on the surface of the rail and wheel, and (ii) variations in the support structure beneath the rail. Thus, vibrations are generated and propagate from the track into the surrounding ground.

There are many parameters influencing the level and characteristics of train-induced vibrations (e.g. Möller et al, 2000; Hall, 2003), such as:

- Vibrations induced by the track structure response
  - Axle load (weight of train and spacing of wheel axles),
  - Geometry and composition of the train (type, cargo, length),
  - Speed of train,
- Wheel-rail interface
  - Wheel defects (eccentricity, imbalance, flats, unevenness),
  - Unsteady riding (bouncing, rolling, pitching, properties of bogie and motor),
  - Acceleration and deceleration of the train,

- Irregularities on the rail
  - Quality of the rail (corrugations, corrosion, unevenness, waviness, joints),
  - Curves and tiling track (centrifugal forces),
- Variations in support structure
  - Geometry and stiffness of the support structure (sleepers, ballast and ground),
  - Frost.

An increase in the axle load will increase the dynamic loading generated by the train. Doubling of the axle load will increase the tunnel vibration levels by 2 to 4 dB (Kurzweil, 1979). The composition of the trains has a great impact on the creation of vibrations, i.e. a train where all wagons are loaded with uniform cargo, e.g., timber, oil or ore generates the largest dynamic disturbance (Möller et al, 2000). Increased train speed will also generate a higher dynamic load. A doubling of the speed will, according to Kurzweil (1979), increase the vibration levels by 4 to 6 dB.

Irregularities on the surface of the rail and wheel are typically seen as waviness. Typical irregularities on the wheel surface are flats generated from locked wheel during breaking, or non-circular wheels. Irregularities on the track can be joints (non-welded tracks), corrugation, waviness, or switches. The presence of wheel flats, loose rail joints or corrugated rail can increase vibration levels by 10 to 20 dB (Kurzweil, 1979). Other sources to vibration at the wheel-rail interaction can be curves, tiling tracks, acceleration and deceleration of the train, and unsteady riding of the vehicle.

Variations in the support structure depend on the geometry, stiffness, and spacing of sleepers. A sleeper may have lost contact with the underlying ballast. It can also be so that one sleeper is better supported and will thus generate a bigger resistance when a train is passing by. It is common that there is a peak in frequency that depends on the spacing of the sleepers and the speed of the train (see e.g. Melke and Kraemer, 1982). Also the stiffness and heterogeneity of the ballast is influencing the characteristics of the forces generated when a train moves along the railway.

As mentioned above, the magnitude of the load generated by trains is due to a static load, which is the weight of the train, and a dynamic load, generated by irregularities on rail, wheels, and substructure. The dynamic loads, which vary greatly depending on when and how they are measured, are added to the static load by the amount listed in Table 2.1. The dynamic and static forces will cause the whole track structure as well as the train to oscillate. This will enhance the stress waves propagating into the ground beneath the track. The generated stress waves all have different characteristics depending on where they are generated, i.e. vehicle, wheel, rail, or substructure.

Table 2.1. Static and dynamic contribution to the load on the rail beneath the train wheel (Sahlin and Sundqvist, 1995).

Type	Load	Size
Static	Weight of train	100 %
Dynamic	Quasi-static contribution in curves	10 – 40 %
	Contribution from uneven rail	50 – 300 %
	Contribution from uneven wheel	50 – 300 %
	Contribution from acceleration and breaking	5 – 20 %

The typical frequency spectra of the vibrations generated by trains in tunnels are from 4 Hz upwards to a few thousand Hz. Typically, there is one or two vibration peaks at different frequencies where the acceleration levels can reach about 80 to 90 dB. Figure 2.3 shows the acceleration levels measured at the track and on the wall of a subway tunnel in Japan. The tunnel is constructed in soil and has a concrete lining with a thickness of about 0.80 m. The speed of the train is about 45 km/h. The maximum acceleration near the track is 80 to 85 dB at 315 Hz, while the maximum is about 60 to 70 dB in the frequency range 31.5 Hz to 200 Hz for the wall. Ungar and Bender (1975) and Kurzweil (1979) have reported similar vibration levels.

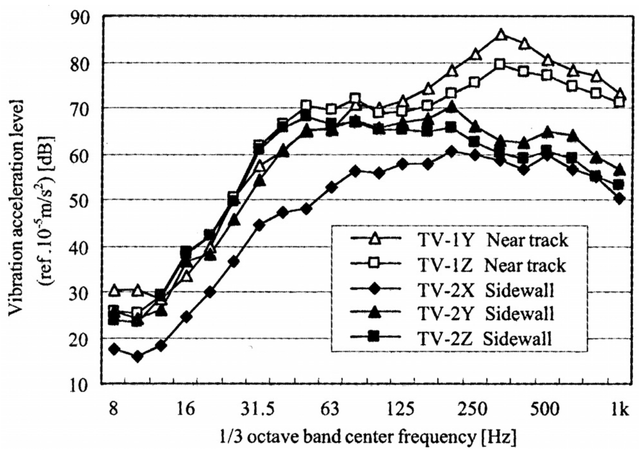


Figure 2.3. Acceleration levels near track and in sidewall inside a tunnel (from Fujii et al., 2005). (X is parallel to the tunnel, Y is perpendicular to the tunnel, and Z is vertical.)

During the propagation the waves will be modified multiple times by geometrical and material variations along its propagation path. In the following sections the propagation path will be briefly reviewed.

2.3 Propagation path

Once the vibration has propagated thought the rail and sleeper they reach the substructure, which normally consists of ballast made of crushed rocks. For normal Swedish train tunnels constructed in rock the ballast is located directly on the rock. If increased damping is needed mats (e.g. ballast mats) can be placed within the ballast. For situations where high attenuation is required, e.g. for tunnels near hospitals, museums, or concert halls, the ballast can be replaced by floating-slabs (mass-spring-systems). As seen in section 2.5, the construction of the substructure highly influences the propagation (attenuation) of train-induced vibrations.

The train-induced vibration propagating through the substructure will, once it reaches the invert, propagate away from the tunnel. The vibration energy will propagate as surface waves along the surface of the tunnel towards the wall and roof, and through the surrounding rock or soil away from the tunnel. Although it is only a short distance, the waves reaching the walls and roof has been attenuated along the propagation path and have lower amplitude than those measured at the invert (Ungar and Bender, 1975, Kurzweil, 1979, NGI, 2004). This is observed for tunnels constructed in both rock and soil. However, the floor and wall of a tunnel driven through rock is believed to vibrate less then a tunnel driven in soil. The vibration levels in a tunnel driven in soil are, according to Ungar and Bender (1975), 5 dB higher at low-frequencies (feelable) and 12 dB lower at high-frequencies (audible) compared to what is observed for a tunnel driven in rock. This is in agreement with findings by Kurzweil (1979) who states that the vibration levels in rock tunnels are typically 6 dB lower compared to soil tunnels. Furthermore, in a Norwegian study it was observed, for the frequency range > 160 Hz, that the measured vibrations at the wall were generated by the noise from the train and not from the train-induced vibrations (NGI, 2004).

For tunnels constructed in soil, and also sometimes when constructed in rock, lining is used to stabilize the tunnel structure. The presence of a lining within a tunnel will attenuate the vibrations and an increase of the lining thickness, and thereby also the mass, increases the attenuation (Kurzweil, 1979; Unterberger *et al.*,1997). A doubling of the thickness will, according to Kurzweil (1979), lead to a reduction of the vibration levels by 5 to 18 dB. The presence of a lining will also affect the propagation of the waves. Ungar and Bender (1975) argued that there is a large transmission loss (low coupling) between the tunnel wall and the surrounding rock or soil for transversal (shear) waves, and that transversal waves hence should be neglected when studying train induced vibrations.

The vibrations reaching the rock or soil will propagate away from the tunnel as elastic waves. These waves will either propagate as body waves (in a body of infinite extent) or as surface waves (half space). Body waves are longitudinal waves, which have a particle oscillation parallel to the propagation direction, and shear waves, which have a particle oscillation perpendicular to the propagation direction. Rayleigh waves, which propagate along a surface, have a particle motion that is generally elliptical in a vertical plane through the propagation direction. Love waves need a thin layer on top of a surface in order to propagate. Both Rayleigh and Love waves are classified as surface waves. See section 3.1 for a more detailed description of the different wave types.

If the ground is assumed to be homogenous the body waves will propagate equally in all directions away from the source. During the propagation the waves would be attenuated from geometrical and material damping. Since Rayleigh waves is propagating along a surface they will be subjected to less geometric damping but will still be subjected to the same amount of material damping as the body waves. The propagation of waves in an elastic homogenous infinite body is well know and fairly easy to mathematically describe. By adding free surfaces the propagation will be more complicated, but can still be described mathematically for some general cases. A more thorough description is given in Chapter 3.

However, in the reality the ground is never homogenous. Usually the ground contains discontinuities of various sizes and shapes, variations in saturation degree, variations in material properties, etc. All these heterogeneities can be seen as boundaries. Also the tunnel, the ground surface, different foundations, etc can be seen as boundaries. Taking account for all these heterogeneities makes it much more complex and it is not longer possible to use simple mathematical relations to describe the wave propagation.

The main objective of this study is to investigate how the rock mass (and soil) influences the propagation of train-induced vibrations. This cannot be covered within this section and has therefore been given a chapter of its own. Hence, the reader is directed to chapter 3 for a deeper insight on wave propagation in rock and soil.

2.4 Receiver

Once the vibrations have propagated through the ground they will eventually reach a receiver, which usually is the foundation of nearby buildings. From the foundation the vibrations will propagate to other parts of the building, causing floors, walls, and ceilings to vibrate. These vibrations can either be felt as whole-body vibrations or heard as low-frequency rumble (structurally radiated noise). The movement of floors and walls can also cause furniture or china to move or rattle, which in turn generate noise, or cause damage to sensitive equipment. In some rare cases train-induced vibrations can also cause structural damage to buildings.

2.4.1 Buildings response to vibrations

The amount of vibrations that is transmitted into the building depends on the coupling between the ground and the foundation. Usually there is a reduction (coupling loss) of the vibrations at the transmission from the ground to the building. Slabs-on-grade are in contact with the underlying soil and will be subjected to similar vibrations as the ground, and the coupling loss is therefore determined to be 0 dB for frequencies lower than the resonance frequency of the slab (Remington *et al.*, 1987). The coupling loss for lightweight buildings is also determined to be 0 (Kurzweil, 1979). For the other foundations types, the coupling loss varies between 2 and 15 dB depending on frequency and foundation type (Remington, 1987; Kurzweil, 1979). For a building supported directly on rock the coupling loss is 0 (Kurzweil, 1979).

The reduction of transmitted vibrations between the ground and building is larger for vertical oscillations than horizontal oscillations since the building is weaker in the horizontal direction. The natural frequency in ordinary dwellings is normally lower than 10 Hz, which is in the same range as for loose soils. Train-induced vibrations are within that range and thus resonance effects are prominent. If the width of a building corresponds to $(n - \frac{1}{2})$ wavelength swaying of the building may occur (Dawn and Stanworth, 1979). If the swaying coincides with the natural frequency of the buildings amplifications may occur.

Once the vibration has reached the foundation they will propagate through the building where the different parts of the building will damp or magnify the vibrations. Ungar and Bender (1975) suggested, based on empirical data, that the vibration as well as the noise level in a room in a multi-storey buildings, L_p, could be estimated by

$$L_p = L_g - 3n \qquad \text{[dB]} \tag{2.1}$$

where L_g is ground vibrations or noise level and n is the number of floors between ground and the room of concern. This gives a reduction of 3 dB between each floor. The reduction is larger for high-frequencies.

Walls, floor and ceiling within a building sometimes works as amplifiers of the vibrations. For lightweight buildings no attenuation is observed, and in some cases the vibration levels on the upper floors has been amplified due to resonance (Kurzweil, 1979). The amplification can vary between 0.5 (reduction) and 2.0 (amplification) within the frequency range 25 to 30 Hz, although amplifications up to 5.0 has been observed (Leventhall, 1987). This is caused by the separate parts having different stiffness, mass and damping which cause them to have different natural frequencies. In Table 2.2 the natural frequencies for common construction components are listed.

Table 2.2. Natural frequencies for different building elements (Leventhall, 1987).

Element	Natural frequency [Hz]
Beams	5-50
Floors and slabs	10-30
Window panes*	10-100
Plaster ceilings in houses	10-20

*Depends on the window size.

Dawn and Stanworth (1979) showed that there can be large variations in the vibration levels as well as in frequency content between two floors within a building. Generally, the amplification is about 5 to15 dB for the frequency range 16 to 80 Hz (Remington, 1987). It is common that the floor amplifies vibration in the 10 to 30 Hz frequency range because the floor resonance frequency coincides with the peaks of the vibrations induced by the train. Kazamaki and Watanabe (1975) observed that the vibration levels were higher in a wooden building right above a tunnel (3 m coverage) than at the nearby ground surface.

Since the floor, walls and ceiling within a building are vibrating noise will be radiated from the surfaces. The sound level within that room depends on the size and shape of the room, the amount of sound absorption in the room, and the vibration levels of the room surfaces (Remington, 1987). To determine the sound pressure level, L_p, within a room Kurzweil (1979) suggested the empirical relation

$$L_p = L_a - 20\log_{10} f + 37 \qquad \text{[dB]} \qquad (2.2)$$

where L_a is the floor acceleration level (in dB) and f is the frequency (see section 3.4.1 for more details). A similar estimate was suggested by Melke (1988), where the sound pressure level, L_p, within the room is determined from:

$$L_p = L_v + 10\lg\sigma + 10\lg\left(\frac{4S}{A}\right) \qquad \text{[dB]} \qquad (2.3)$$

where L_v is the vibration velocity level of the surface (in dB), σ is the radiation efficiency, S is the area of the vibrating surface, and A is the absorption area of the room. As discussed in section 2.4.2 radiated noise can, if levels are high enough, cause annoyance among the occupants.

Vibrations, regardless of source, e.g. earthquake, blasting, traffic or trains, can cause damage to buildings. Train-induced vibrations can cause damage to buildings in the form of (i) strain, (ii) natural vibrations, or (iii) settlements (BV, 1997). Strain can be caused by deflection from the train if the track is close to the building. It can also be caused by the stress wave propagating along the ground surface. As mentioned above, if the train-induced vibrations have a frequency that is near the natural frequency of the building, resonance of the building may arise. The vibrations have to have a reasonable duration for resonance to occur and can generally only be caused by freight trains that are uniformly loaded. For certain soils train-induced vibrations can cause and/or accelerate settlements. Since there are many factors that can contribute to settlements it is usually difficult to determine what part the vibrations is responsible for. Train-induced vibrations can in extreme cases trigger slides, but is never the sole cause.

Leventhall (1987) has classified damage to building into three categories:

- *Minor damage* (or architectural damage) – Results in cracks of a few mm in width in plasters, or loosening or dislodgment of tiles, etc. Cosmetic repairs are only needed.
- *Major damage* – Results in cracks in walls and lintels. Can be up to 10 mm in width. Can also result in plaster falling from the ceiling, etc. Professional repair is needed.
- *Sever damage* – Result in cracks about 25 mm wide. Can lead to potential destruction of a building. Major repair work is needed for the building to maintain its habitability.

The normal causes to damage of buildings are from thermal effects, expansion due to moisture, different settlements of soft ground, frost heave in soils, shrinking and expansion of clay, chemical affects, nearby trees, etc. (Leventhall, 1987). For old buildings modifications made, e.g. creating opening in walls, together with deterioration of the strength can cause damage. Failure in newer buildings is often caused by unauthorized modification or faults. Consequently, it is not likely that train-induced vibrations will cause damage to buildings. If a building would be damaged from train-induced vibrations it is usually caused in combination with other factors, e.g. alteration of ground water level, which would have caused damage to the building regardless of the presence of vibrations. The vibrations merely accelerate the process.

The damage potential of buildings depends on the age, size, fatigue properties, structural resonance, and type of construction. In the review by Leventhall (1987), it was determined that the safe limit for residential buildings is 50 mm/s (ppv). The threshold for architectural damage is 5 mm/s (ppv), while for old and historical buildings the threshold is 2 mm/s (max). The vibration levels required to cause damage to buildings are generally much higher than what humans consider tolerable. Therefore, the acceptable limits for humans will be the limit for allowable vibrations in buildings (see Section 2.4.2).

Nevertheless, people commonly accuse vibrations to cause cracks whiting their dwellings although the vibration levels are rarely high enough to be the cause. Many people associate noise with vibrations, and when hearing loud noises this makes them inspect their properties. Another aspect is, according to Dowding (1996), that it is easier to get economical compensation for physical damage of your properties, than for annoyance (psychological damage).

2.4.2 Human response to vibrations

Humans can apprehend vibrations in two ways, either as (i) perceptible vibrations or as (ii) audible sound. Perceptible vibrations are those that are felt since a part of the body is in contact with a vibrating surface. Audible sound can either be low-frequency rumble or rattling windows or china, where both types are caused by vibrating floor and walls. How humans respond to vibrations and noise depends on their activity but also on the magnitude and frequency content of the vibration. If levels are found to be too high annoyance is the normal result.

Perceptible vibrations

The human body is affected by vibrations of any frequency if the amplitude is large enough. When studying the human behaviour considerations for mechanical as well as physiological effects have to be addressed. The aspects of interest are (i) the characteristics of the body while subjected to vibrations, (ii) the effect of the disturbances (physical, physiological and psychological), and (iii) acceptable exposure levels for certain exposure times and frequencies (Bodén *et al.*, 2001).

Perception threshold

The perception threshold is the lowest level at which a human can feel a vibration. This threshold is highly individual and depends on the psychological condition of the human. Also the task or activity the person is occupied with influences the threshold. Moreover, the perception level is also dependent on the frequency of the vibration. The perception threshold for an alert and focused person is about 0.01 mm/s^2 (rms) at low frequencies (1 Hz) and increases with frequency to about 0.1 mm/s^2 (rms) at 100 Hz (Griffin, 1990). If the velocity of the vibration is measured instead the perception threshold is about 0.1 to 0.3 mm/s (rms) within the frequency range of 10 to 100 Hz (BV 1997). Parameters influencing the human perception threshold to vibrations are according to Pretlove and Rainer (1995):

- position (standing, sitting, lying down),
- direction of incidence with respect to spine,
- personal activity (resting, walking, running),
- sharing the experience with others,
- age and sex,
- frequency of occurrence and time,
- the character of the vibration.

Old people have a higher perception threshold. Men have a lower threshold than women for some frequencies. The perceptible threshold of vertical vibrations for a standing and a sitting individual is similar for most situations. Horizontal vibrations are, at low frequencies (1 to 10 Hz), perceptible at similar thresholds as those for vertical vibrations. However, at higher frequencies the perceptible threshold increases due to reduced transmission of horizontal vibrations to the body (Griffin, 1990). The perception threshold of horizontal vibrations is similar for a standing and a sitting person except for frequencies between 1 to 16 Hz where the threshold is higher for a standing person. The perception level for a person lying down is independent of frequency (within the range 1 to 100 Hz) of vertical vibrations, while the perception level increases with increased frequency for horizontal vibration. If a person is occupied with a distracting activity the perception threshold will increase, while having a visual reference will decrease the perception threshold. However, the greatest difference in perception threshold is observed between individuals and not by various factors.

Effects

The effects of vibrations can be grouped in three criteria; (i) health, (ii) comfort and sensation, and (iii) motion sickness (Bodén *et al*., 2000). Physiological effects from vibrations can appear as increased heart activity, and increased pulse and breathing. The train-induced vibrations in buildings are small and will therefore not cause any permanent physiological (health) effects on humans nor will it affect everyday activities (BV 1997). Activities that commonly are affected by vibrations are sleep (falling a sleep as well as maintain being asleep), concentration problems, speech interference, and decreased work capacity. However, the vibration will generate annoyance among the residents, which may lead to complaints. Motion sickness is caused by low-frequency (about 1 Hz) vibrations when travelling and can therefore be disregarded when studying train-induced vibrations.

Acceptable exposure levels

Pretlove and Rainer (1995) summarized perceptibility thresholds from various sources and found that for 1 to 10 Hz perceptibility is proportional to acceleration, whilst for 10 to 100 Hz the perceptibility is proportional to the velocity. Different perceptibility thresholds are shown in Table 2.3. Measurements conducted by Banverket (1997) show that at a level of 0.5 mm/s vibrations are definitely perceptible. At levels above 1.2 to 1.5 mm/s at night most people determine the vibrations as clearly perceptible. People sleeping lightly can be awakened at levels of 1.5 to 2.0 mm/s. People being used to vibrations generally have a higher threshold then people not used to the disturbance. National and international codes and regulations regarding acceptable exposure levels to vibration are presented in section 4.2.

Table 2.3. Human perceptibility thresholds for vertical harmonic vibrations for a person standing (Pretlove and Rainer, 1995).*

Description	Peak acceleration [mm/s^2] for 1 – 10Hz	Peak velocity [mm/s] for 10 – 100 Hz
Just perceptible	34	0.5
Clearly perceptible	100	1.3
Disturbing/unpleasant	550	6.8
Intolerable	1800	13.8

*) There is a scatter by a factor of up to about 2 on the given values.

Audible sound

Once the ground-borne vibration has been transmitted into a building the different construction elements will vibrate. Walls, floors, and ceilings that vibrate may radiate audible noise. This can either be low-frequency rumbling noise from the vibrating construction elements, or as secondary noise, i.e. high-frequency noise radiating from internal decorations and furnishing, e.g. windows or china, put into vibration by rattling construction elements. Acceptable exposure levels of noise according to national and international codes and regulations are presented in section 4.1.

Effects

Noise consists of unwanted sounds of various kinds and is very subjective – something that is considered as noise by one listener can be regarded as desirable by another, e.g. music – people therefore reacts differently to noise. Someone that is subjected to noise may be affected both physiologically and psychologically. Common physiological effects are contraction of blood vessels, increase of the pupil size, and changed breathing (Bodén *et al*., 2001). Being subjected to sound levels that are high can cause temporary loss of hearing which, if persistent long enough, can lead to permanent hearing loss. Noise reduces attentiveness and may thereby influence the work performance; not necessarily by rate of work but by reducing the accuracy of the performance. Noise can also interfere with communication and sleep.

The sound level, frequency content, and duration will influence how humans apprehend noise. In general, noise that is fluctuating, heard at night, or repeated often (like that passage of a train), is considered as more annoying (Davis and Cornwell, 1998). Noise combined with perceptible vibrations will lower the level at which the noise will be considered as annoying (Howarth and Griffin, 1990). Unexpected noise with short duration is usually not considered as annoying. Train-induced noise will not cause hearing loss or other physiological damage to humans in dwelling above tunnels. Thus, annoyance is the major aspect that has to be considered within this study.

2.5 Countermeasures

As seen in the previous sections, vibrations (and noise) will foremost cause annoyance among residents along the railway, while damage to structures only occur in rare cases. In order to decrease the vibration levels in dwellings, various countermeasures can be applied. Today there exist several measures to reduce vibration generated by trains moving in tunnels. The different measures can be applied at any position along the propagating path, i.e. either at the source, along the path, or at the receiver. However, according to Deischl *et al*. (1995) and Kazamaki and Watanabe (1975) it is more effective and economically beneficial to perform reducing measures at the track (source).

The most common measures that can be applied at the source in order to reduce emissions from train traffic on open tracks as well as in tunnels are (Hemsworth, 2000; Deischl *et al*., 1995):

- Rail surface
- Rail pads
- Rail fastenings
- Ballast thickness
- Ballast mats
- Mass-spring system

Applying a measure in order to reduce vibrations is more about alteration of commonly used solutions i.e. by changing material used or the thickness of the ballast, than inventing new revolutionary solutions. Mass-spring system is an example of the latter. Some of the common measures to reduce the train-induced vibrations are discussed below.

Rail surface

The quality of the surface of the rail is very important with regard to train induced vibration, but also for the comfort of the passengers. Various irregularities, such as short and long pitch corrugations, insulating joints, turnouts, etc. will appear along the rail from the numerous passing of trains. Kazamaki and Watanabe (1975) observed that there was a difference of 10 dB between new rails and wheels compared to corrugated rails and wheels with flats from normal service wear. In order to reduce the vibrations it is therefore important to maintain the rail in good condition, or even use high-strength steel instead (Deischl *et al*., 1995). Thus, having a good maintenance program for the rail can be seen as an important and a good measure to reduce vibrations (Deischl *et al*., 1995; Kazamaki and Watanabe, 1975).

Rail pads

Rail pads, sometimes also known as "sole" plates or pads, are placed between the rail and the (concrete) sleeper. They are usually made of rubber and their main function is to reduce fatigue cracking of the sleepers, but they are also believed to have a damping effect on vibrations. The measure here is either to install the pads or to use pads with a different stiffness. For ballasted tracks this measure has been determined to be ineffective in the reduction of vibrations (Hemsworth, 2000).

Rail fastenings

Rail fasteners are used to keep the rail at its designated position on the sleeper. There are various variants that are optimal for different conditions. Using highly elastic (flexible) rail fastenings will permit larger deflections of the rail beneath the wheel which reduces the mechanical impedance of the superstructure and hence the vibrations (Deischl *et al*., 1995). Using flexible fastenings reduces the vibrations between 30 to 50 Hz, where a higher reduction is observed at 50 Hz (about 6 to 10 dB).

Ballast thickness

The main purpose of the ballast is to distribute the pressure from the track. It also provides a foundation for the sleepers holding them in position. Moreover, it has a draining purpose. The normal height for the ballast is about 0.3 m. An increase of the thickness has no measurable effect (Hemsworth, 2000), while a decrease in thickness leads to deterioration of the attenuation (Deischl *et al*., 1995). The Norwegian Geotechnical Institute (NGI, 2004) observed that an increase of the ballast thickness (1 m) increased the attenuation; however, it was believed that the thicker structure gave a greater load distribution and the reduction was concluded not to be caused by damping solely. It has also been observed that newly tampered ballast generates greater attenuation than ballast not tampered for a long time. This, along with the importance of a smooth rail surface, implies that maintenance is an important aspect of reduction of train-induced vibrations.

Ballast mats

Ballast mats, or sub-ballast mats, are, as the name implies, elastic layers that are placed beneath or inside the ballast bed. Ballast mats (thickness up to 80 mm) are considered to have high efficiency to attenuate vibrations within the range 16 to 50 Hz where a reduction as high as 20 dB can be reached at 50 Hz (Deischl *et al*., 1995). Kazamaki and Watanabe (1975) reported a reduction of 5 to 8 dB due to the use of ballast mats. One type of ballast mat applied on concrete base generated reduction of about 10 dB for frequencies above 40 Hz (Nelson, 1996). Placing the ballast mat higher up within the ballast results in higher attenuation (NGI, 2004). If the thickness of the ballast is increased from 0.3 to 0.6 m in combination with sub-ballast mat, a reduction of 4 dB can be added. It should be noted that the increase in ballast only have an effect if there is a sub-ballast mat installed (Deischl *et al*., 1995).

Mass-spring system

Mass-spring system, or floating-slab-system, is the most effective measure for train-induced vibrations in tunnels (Hemsworth, 2000; Deischl *et al*., 1995). The principle idea is to have a linear harmonic oscillator that has a very low natural frequency. Usually the oscillator is a heavy concrete slab that is isolated from the tunnel invert by rubber bearings or steel springs. A floating slab should have as low natural frequency as possible in order to attenuate the vibrations to as large extent as possible. It is not practically possible to have a natural frequency lower then 5 Hz; neither should it exceed 14 Hz (Deischl *et al*., 1995). Normally the natural frequency is between 8 to 12 Hz. Hemsworth (2000) reported 10 dB attenuation at 16 Hz and 25 dB at 125 Hz, while Kazamaki and Watanabe (1975) reported attenuation levels between 15 to 21 dB. However, Hunt (2001) showed, with the aid of numerical analysis, that if the natural frequency of the floating-slab system is not low enough, the attenuation effect would be diminishing.

Since most tunnels are unique the required attenuation varies and the slabs are designed to fit with the cross-section of the tunnel. The slabs require a height of 0.8 to 1.4 m and can weigh between 4 000 and 9 000 kg/m. Floating slabs can be used for tracks both with and without ballast. A negative aspect with floating-slab-systems is that the system is more expensive than the other systems used to reduce vibrations.

It is important that the bearings (or springs) can handle the load efficiently. If the damping of the bearing is neglected, the natural frequency of the system can be determined from the relation (Deischl *et al*., 1995)

$$f_0 = \frac{5}{\sqrt{w}} \tag{2.4}$$

where w is the dead load deflection in cm (deflection from slab and superstructure). As illustrated by Deischl et al. (1995) a 3 mm deflection will give the slab and the superstructure a natural frequency of 9.1 Hz. However, due to nonlinearity in the system the natural frequency is normally 10 – 20% higher. Adding the deflection from the train should not generate a deflection that is twice as large as the static deflection.

Other methods

There are other methods available for reducing the vibrations except those mentioned above. One method is to apply damping material to the rail web. (The rail web is the middle section of a standard rail.) This will reduce the noise level in a tunnel with about 2 to 5 dB (A). Due to the low reduction it cannot be considered as a cost-effective measure (Deischl et al., 1995). Another method is to apply pads between the sleepers and the ballast. This gives an attenuation of 15 dB at 125 Hz when used on a ballasted track (Hemsworth, 2000). Kazamaki and Watanabe (1975) installed vibration proof sleepers in a 2 km tunnel in Japan and observed a reduction of vibration of about 6 to 14 dB.

Applying measures along the propagation path (i.e. ground) is not common due to the difficulty and the high costs involved. For open track railways barriers or trenches are often used. Barriers are fences of various kinds that are placed along the railway and their main purpose is to reduce the noise generated by the train. Trenches on the other hand is used to reduce the vibrations and can be either open or in-filled. Their attenuating abilities depend on size (width and depth), distance from building, and whether they are filled or not. Through numerical analysis Adam and Estorff (2005) found that wider and deeper trenches attenuate more vibrations. Open trenches gives higher attenuation compared to in-filled trenches (Hung et al., 2004). Increased distance between trench and building has a negative effect on the attenuation and can sometimes be eliminated (Adam and Estorff, 2005). Nevertheless, trenches are not a good solution for trains moving underground unless they are located near a building where they may have some reducing effects.

One measure to reduce vibration would be to increase the thickness of the lining. From numerical simulations Unterberger *et al*. (1997) found that thickening of the lining will reduce the vibration of the lining itself. When the overburden is small (1 to 2 tunnel radii) thickening will also reduce the vibration at the foundation level (2.5 m below surface). For greater depth the reduction was found to be small. However, higher frequencies are reduced at the foundation level, while the lower frequencies are enhanced. Due to the low natural frequencies of buildings this is a negative effect of lining thickening. Hence, Unterberger *et* *al*. (1997) concluded that thickening of the lining should only be an option when (i) the tunnel and building foundation is in direct contact, (ii) the tunnel is less then 1.5 diameters below the building foundation, or (iii) when the natural frequency of the building is exactly known.

Another measure would be to increase the weight of the tunnel (which in a way is similar to thickening the lining). Kazamaki and Watanabe (1975) increased the floor slabs from 0.6 m to 2.0 m (and thereby the tunnel weight) and observed a 5 dB reduction of the vibration levels. A rough estimate was therefore introduced stating that doubling the weight of the tunnel will reduce vibrations by 5 dB. Unterberger *et al*. (1997) simulated an added weight below the invert of a lined tunnel situated in soil. It was found that the vibration of the lining was reduced 7 to 10 dB and that the vibrations at a foundation on the surface were reduced between 4 to 10 dB (depending on depth of tunnel).

Kazamaki and Watanabe (1975) investigated how the constraints of the ground influenced the vibration in a train tunnel. When the surrounding soil was removed it was observed that the vibration levels in the roof, wall and floor was 2 to 5 times higher than when the tunnel was constrained from the soil. The predominant frequency, for the unconstrained case, ranged from 40 to 85 Hz. It was hence concluded that the condition of the surrounding soil has a great impact on the vibrations of a tunnel.

Reducing the vibration at the receiver is complicated and therefore also expensive. Usually it is the foundation of the building that is isolated. One method it to treat the building as a rigid body supported by springs and dampers (the basics are similar to that of floating-slab system). Such a system can reduce vibrations above 10 Hz significantly.

It is easier and less expensive to apply the measures at the source for a railway constructed in an urban area, than it is to apply individual measures in all the buildings along the railway, in order to fulfil the vibration requirements. However, for particularly sensitive buildings (hospitals, theatres, etc.) or in cases when there are only few buildings along the tunnel, individual measures should be applied to fulfil the requirements.

3 WAVES AND THEIR BEHAVIOUR IN GEOMATERIALS

From a train moving through a tunnel vibration will radiate from the invert of the tunnel out into the rock mass and the soil as waves. Since the objective of this study is to determine how the properties of the rock mass (or soil) will influence train-induced vibrations how waves behave and propagate in rock masses is essential. Consequently, within the following sections wave propagation in general will be reviewed.

Firstly, wave propagation in isotropic, homogeneous and elastic medium will be briefly reviewed. Wave propagation in rock and soil is thereafter reviewed, where the emphasis will be on propagation in rock. Finally, different methods used to estimate train induced vibration from underground tunnels will be reviewed.

3.1 Elastic waves

3.1.1 Body waves

When an isotropic, homogeneous and elastic medium is subjected to a force, particle displacement arises and two types of waves are generated; (i) *longitudinal* and (ii) *shear* waves. These waves radiate away from the source and propagate in all directions. If the waves propagate in a homogenous media, i.e. no boundaries present, these two waves are the only waves generated.

Longitudinal Waves

Longitudinal waves have the highest propagation velocity and will be the first wave to arrive. For a longitudinal wave the particle movement is parallel to the propagation direction. The propagation velocity for a longitudinal wave, c_p, can, for an isotropic unbounded medium, be determined from the relation

$$c_p = \left[\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)} \right]^{\frac{1}{2}}, \tag{3.1}$$

where E is Young’s modulus, ν is Poisson’s ratio and ρ is the density of the material in which the wave is propagating. Compression wave can also be called *dilatational*, *compression* or *P wave*.

Shear Waves

Shear waves are slower than longitudinal waves. For shear wave the particle movement is perpendicular to the propagation direction. The propagation velocity of shear waves, c_s, in an unbounded medium can be determined from the relation

$$c_s = \left[\frac{G}{\rho}\right]^{\frac{1}{2}}, \tag{3.2}$$

where the ρ is the density, and G, is the shear modulus of the medium in which it is propagating. Since shear waves move by shearing the material it cannot propagate through air and water. Shear wave can sometimes be called *distortional*, *transversal* or *S wave*.

The displacement caused by S waves can have any direction in a plane normal to the direction of propagation. Usually, in order to simplify only two directions are considered; vertically (SV) and horizontally (SH) polarized S waves (Das, 1993), as shown in Figure 3.1.

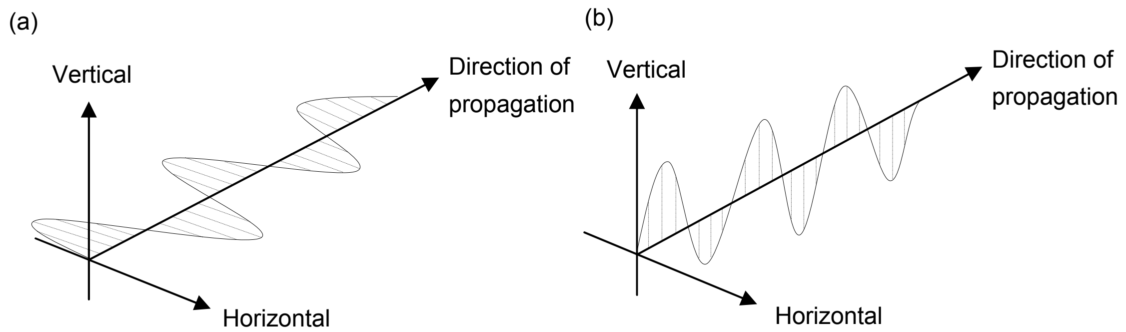


Figure 3.1. The two directions of transversal waves; (a) horizontal polarized (SH) waves and (b) vertically polarized (SV) waves.

The ratio between the velocity of P and S waves, κ, is not constant and varies according to the relation

$$\kappa = \frac{c_p}{c_s} = \left[\frac{2(1-\nu)}{1-2\nu}\right]^{\frac{1}{2}}, \tag{3.3}$$

where ν is Poisson's ratio. Hence, the relation between P and S waves is a function of Poisson's ratio, where an increase generates a higher ratio (as seen in Figure 3.2). Generally the ratio is set to 1.7 ($\nu = 0.25$).

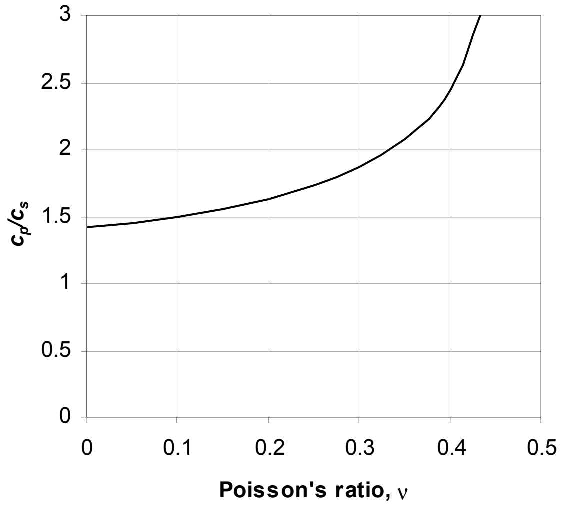


Figure 3.2. Variation of the ratio between P and S wave velocities as a function of Poisson's Ratio (after Das, 1993).

3.1.2 Surface waves

For a bounded medium, which has a free surface of some kind, surface waves will be generated. They originate from body waves that are interacting along boundaries. Hence, surface waves will always appear along the ground surface and along discontinuities. The most common surface wave is the *Rayleigh* wave, which propagates along the surface. If there is a layer with different properties covering the bounded medium, *Love* waves can be generated within the layer. Another surface wave is the *Stoneley* waves which propagate along the surface between two solids that has different mechanical properties.

Rayleigh wave

The most known surface wave is called Rayleigh wave (R wave), and consists of both longitudinal and lateral movements. The propagation velocity is smaller than that of body waves. If the S wave velocity, c_s, is known the R wave velocity, c_r, can be determined with good accuracy from the relation (Achenbach, 1973)

$$c_r = \frac{0.862 + 1.14\nu}{1+\nu} c_s \qquad (3.4)$$

where ν is Poisson’s ratio. Varying Poisson’s ratio from 0 to 0.5 increases the R wave velocity from $0.862c_s$ to $0.955c_s$.

R waves do not propagate far into the medium, since the velocity decreases with increased depth, and at a depth of 1 to 2 wavelengths, the velocity is negligible. The amplitude of the wave first increases with depth, reaches a maximum at a depth of 0.2 to 0.6 wavelengths, and thereafter decreases to a very small value at a depth of 1.3 wavelengths.

Love Waves

If there is a thin layer on a half space, Love waves (L wave) can occur. The layer has to have a density and elasticity that differs from those of the half space, and that result in a S wave velocity that is lower than the velocity in the half space. The velocity of L waves increases with increased wavelength e.g. the wave is dispersing. L waves are horizontally polarized S waves (SH wave) that are reflected between the top and bottom within the layer. A L wave will not propagate into the half space.

Stoneley Waves

In the surface between two half spaces there are waves generated that are dependent on the properties of both solids. If the transversal wave velocities in both media are nearly equal, Stoneley waves may occur. They propagate faster than R wave but slower than S waves.

3.1.3 Refraction and reflection of waves

When body waves (P and S waves) impinge a *boundary between two solids* with different physical properties, both reflection and refraction will take place. Generally, one of each wave type is reflected back into the first medium while one of each wave type is refracted into the second medium. If the second media cannot transport mechanical waves, e.g. vacuum, a *free surface* is generated and there will only be reflected waves.

For a P wave (P_1) impinging a *boundary between two solids*, as shown in Figure 3.3a, there will be two reflected waves and two refracted waves. The reflected waves consist of one P wave (P_2) and one vertically polarised S wave (SV_1), which will propagate through layer 1. In layer 2 the refracted waves, consisting of one P wave (P_3) and one vertically polarised S wave (SV_2), will propagate. If Huygens's principle can be applied to the waves, the relation between velocities and angles can be stated as (Kolsky, 1963)

$$\frac{\sin\alpha_1}{c_{P1}} = \frac{\sin\alpha_2}{c_{P1}} = \frac{\sin\beta_1}{c_{S1}} = \frac{\sin\alpha_3}{c_{P2}} = \frac{\sin\beta_2}{c_{S2}} \tag{3.5}$$

where c_{p1} and c_{p2} are the P wave velocities in the first and second media, respectively, and c_{SV} and c_{S2} are the S wave velocities in the first and second media, respectively. The angles are explained in Figure 3.3a. If the P wave impinges normal to the surface (α_1 = 0°) there is no reflection of S waves.

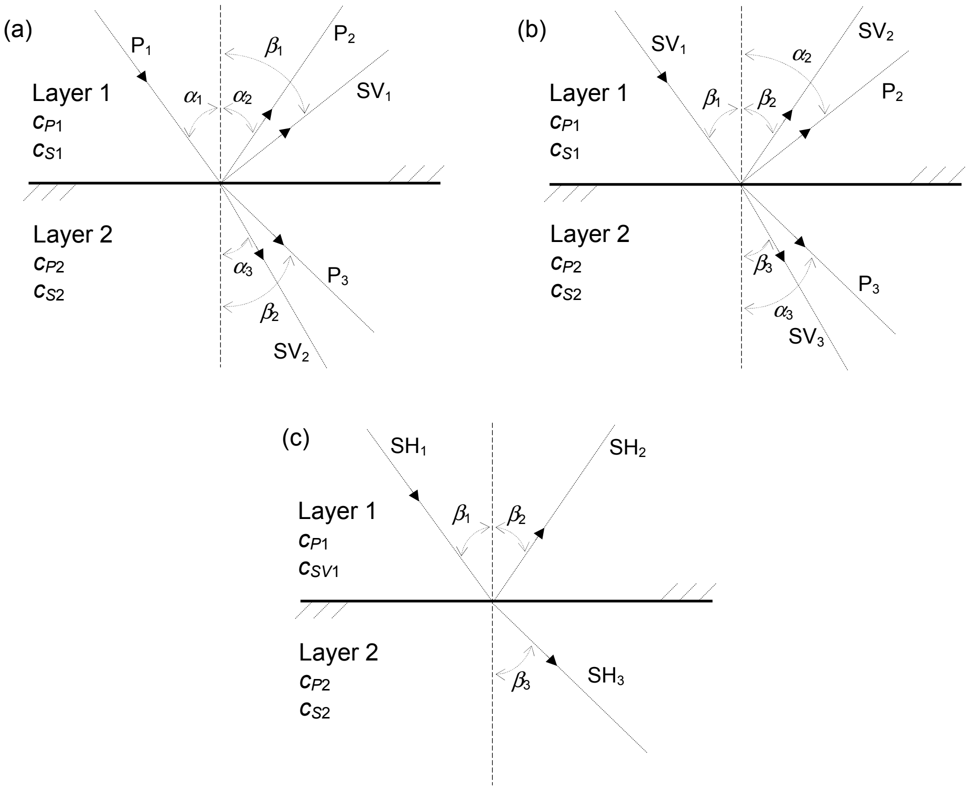


Figure 3.3. Reflection and refraction of P and S waves. (a) An incident P wave is reflected and refracted, (b) an incident SV wave is reflected and refracted, and (c) an incident SH wave is reflected and refracted. If layer 2 is vacuum, there will be a free surface and no refraction will occur.

When a vertically polarized S wave (SV) impinges a *boundary between two solids*, as shown in Figure 3.3b, there will be two reflected waves and two refracted waves. The reflected waves consist of one vertically polarised S wave (SV_2) and one P wave (P_2), which will propagate through layer 1. In layer 2 the refracted waves, consisting of one vertically polarized S wave (SV_3) and one P wave (P_3), will propagate. This gives

$$\frac{\sin\beta_1}{c_{S1}} = \frac{\sin\beta_2}{c_{S1}} = \frac{\sin\alpha_2}{c_{P1}} = \frac{\sin\beta_3}{c_{S2}} = \frac{\sin\alpha_3}{c_{P2}}\,. \tag{3.6}$$

For the case where a horizontally polarized S wave (SH_1) impinging a *boundary between two solids*, as shown in Figure 3.3c, only horizontally polarized S waves will be reflected (SH_2) and refracted (SH_3). This gives

$$\frac{\sin\beta_1}{c_{S1}} = \frac{\sin\beta_2}{c_{S1}} = \frac{\sin\beta_3}{c_{S2}}. \tag{3.7}$$

At a *free surface* (waves cannot propagate in the second medium) the amplitudes of the reflected and refracted waves depend on the material constant κ (Eq. 3.3), the incidence angle of impinging waves, and the angles of reflection/refraction. At an interface between two media the amplitude depends on the mechanical impedance (the product ρc) of the two media. The relations for determine the amplitudes become complicated and the reader is referred to Achenbach (1973) and Miklowitz (1978) for further details.

3.2 Attenuation

When a wave propagates through any physical medium, energy is lost to the surrounding medium, i.e. the wave is attenuated. This leads to a gradual decrease of the amplitude with distance travelled. Usually attenuation is separated between (i) *geometrical* and (ii) *material damping*.

3.2.1 Geometrical damping

Geometrical damping is caused by spreading of the waves and depends on the type of wave propagating through the material. Attenuation from geometrical spreading is frequency invariant (i.e. not frequency dependent). For body waves propagating with hemispherical wave front, the amplitude, A, of the wave decreases proportionally to the radius, r (distance to source), as

$$A \propto \frac{1}{r}. \tag{3.8}$$

For surface waves propagating with cylindrical wave fronts, the amplitude decreases as

$$A \propto \frac{1}{r^2}. \tag{3.9}$$

The amplitude of the R wave, which also propagates with cylindrical wave front along a free surface, is less attenuated compared to the other wave types (Das, 1993). The amplitude is govern by the relation

$$A \propto \sqrt{r}\ . \qquad (3.10)$$

3.2.2 Material damping

Material damping is caused by loss of energy to the material in which the waves are propagating (usually due to friction). The energy (amplitude) decreases exponentially and is different for different materials. Since the R wave is a surface wave it suffers less geometric attenuation compared to body waves but is still subjected to material damping. Due to the lower attenuation R waves will carry most of the vibration energy at distances far from the source.

3.3 Geomaterials

For the ideal case, i.e. where the ground is homogenous, the vibrations (waves) propagate in all directions away from the source. During their propagation they are subjected to attenuation due to geometric spreading and dissipation (material damping). It is obvious from practice that the ground is not homogeneous. Usually the ground consists of discontinuities of various sizes, different soil and rock types, and different interfaces. Geomaterials can also be stratified and contain water. The heterogeneities will generate additional modes of vibrations which can propagate along interfaces (joints, boundaries, etc.). Waves can also be subjected to mode conversion from one kind of wave to another (as at boundaries).

3.3.1 Intact rock

There are several physical properties that influence the propagation velocity of elastic waves in a rock mass. According to Lama and Vutukuri (1978) these are:

- rock type,
- texture,
- density,
- porosity,
- anisotropy,

- stress,
- water content, and
- temperature.

With regard to the scope of this study it is evident that some of the above listed properties will not significantly influence the vibrations generated by passing trains. Those of interest for this study are briefly reviewed below.

Rock type

The propagation velocity is higher for dense and compact rocks and lower for rocks that are less dense and less compact (Lama and Vutukuri, 1978). For some rock types the range of the propagation velocity is larger than for other rocks. For example, in a limestone the P wave velocity can range from 2.0 and 6.0 km/s due to variations in texture. Granite and sandstone both contain silica but since their structure differs the P wave velocity between the rock types also differs; 5.0 km/s and 3.0 km/s, respectively.

Texture

The velocity in rock depends on the velocities in the different mineral components. Increasing quartz content will decrease the velocity and increased hornblende content will increase the velocity (Ramana and Venkatanarayana, 1973).The grain size also affects the wave velocity; larger grains will decrease the velocity and finer grains will increase the velocity (Lama and Vutukuri, 1978).

Density

The velocity of P waves increases as the density increases (Ramana and Venkatanarayana, 1973). The velocity of the S wave also increases with greater density, but to a smaller extent compared to the P wave velocity.

Porosity

When the porosity increases the velocity generally decreases (Lama and Vutukuri, 1978). Tests conducted by Youash (1970) and Ramana and Venkatanarayana (1973), with the use of elastic waves, show a general decrease even though the values are scattered.

Stresses

With increased stress, the P and S wave velocity will increase (Lama and Vutukuri, 1978). The increase is larger at low stresses than at high stresses. It follows from the decrease of porosity, closure of cracks and an increased contact between grains due to tighter interlocks. Hence, the influence on wave velocity due to increased stresses is more prominent for porous and loose rocks than for compact rocks. The velocity is higher when unloading than when loading, since some closed cracks remain closed during unloading (Lama and Vutukuri, 1978).

Water Content

The water content affects the wave velocities, especially in porous rocks (Lama and Vutukuri, 1978). If the P wave is assumed to propagate through the mineral skeleton and through the pores, the velocity can be determined according to the relation

$$c = \frac{c_f c_m}{c_m n + c_f (1-n)} \tag{3.11}$$

where n is porosity, c_f is the elastic wave velocity in the substance filling the pores and c_m is the elastic wave velocity in the mineral skeleton of the rock. If the medium filling the pores has a higher wave velocity than air, it leads to higher total velocity. Since the velocity in water is five times higher than in air, the velocity will increase for saturated hard rock (Lama and Vutukuri, 1978; Paterson, 1978). For more porous rock saturated with water, the velocity (inversely proportional to the porosity) will be lower than for less porous rocks, because the velocity for water is lower than the velocity for the mineral skeleton. The velocity for S waves is almost insensitive to water saturation because the waves can only pass through the mineral skeleton.

3.3.2 Rock mass

Anisotropy

A large quantity of the rock that is present in the earth's upper crust is anisotropic, i.e. has different properties (physical, dynamic and mechanical) in different directions. A rock mass can be anisotropic due to layers of different rock types i.e. sedimentation. Also a rock mass cut by one or several regularly spaced joints can be defined as anisotropic in addition to being discontinuous. Also the rock types may be anisotropic due to sedimentation or the presence of cracks. Sometimes several types of planar anisotropy can be present within a rock mass at the same time e.g. joints and foliation or joints and bedding planes (Amadei, 1996).

The crack density affects the P and S wave velocity. This decrease is valid in all directions, but only if there is no preferred orientation of the cracks. When the cracks have a preferred orientation, velocity anisotropy will occur, e.g. different velocities in different directions (Paterson, 1978). The velocities of the wave propagating normal to the plane, in which the cracks are oriented, are strongly influenced. If the wave propagates in the plane, the influence on the velocity is small. The coefficient or degree of anisotropy, k, is often defined as the ratio between the velocity parallel, $c_{parallel}$, to the planes and the velocity perpendicular, $c_{perpendicular}$, to the planes (Lama and Vutukuri, 1978). It can be written as

$$k = \frac{c_{parallel} - c_{perpedicular}}{c_{perpendicular}} \cdot 100 \tag{3.12}$$

and is expressed in %.

Joints

As described in Chapter 3.1, a wave, either a P or S wave, encountering a discontinuity will be either reflected and/or refracted. At a free surface (i.e. ground surface) only reflection takes place while at a boundary (joint or interface between two rock types) both refraction and reflection occur. A wave propagating along a discontinuity between two materials with different physical properties (and thereby different wave velocities) will propagate with the higher of the two velocities.

Fractures of various sizes ranging from a few µm (micro cracks) to tens for meters (faults) are a natural part of a rock mass. Their presences will affect the waves propagating through the rock mass in the form of attenuation. If a discontinuity (fracture) is present within an elastic, homogenous and isotropic medium, a part of the energy from the impinging wave will be reflected, while the rest will be transmitted across the fracture, as illustrated in Figure 3.4. This is done in the same way as for waves impinging a boundary between two solids (see section 3.1.3).

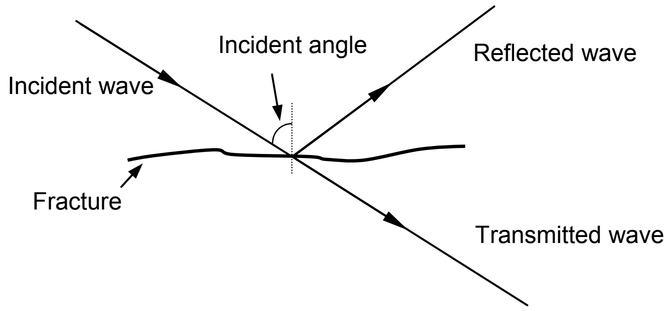


Figure 3.4. A simple illustration of a wave propagating across a discontinuity (after Boadu and Long (1996)).

It is therefore of interest to know the amount of energy (amplitude) that is attenuated when the wave propagates across a joint. Joint properties influencing the propagation (e.g. affecting the amplitude) of waves are (Boadu and Long, 1996; Fratta and Santamarina, 2002):

- size or length,
- aperture,
- contact area of the fracture surfaces,
- filling material,
- normal stresses, and
- orientation with respect to the incident wave (angle of incidence).

Also properties of the incident wave, such as velocity and frequency, influence the propagating across a fracture. For the general case, where only one joint is present, the amplitude of reflected P and SV waves are significantly reduced (incident angle was 30°). This decrease is not observed for the reflected SH wave. The transmitted P and SH wave maintains a similar pulse shape as the incident wave (e.g. only a minor amplitude decrease), while the amplitude of the transmitted SV wave is heavily reduced (Boadu and Long, 1996). Also the frequency content is affected, where the most significant decrease is observed for high frequency reflected P, SV and SH waves.

Increased fracture length will result in decreased velocities for P, SV and SH waves (Boadu and Long, 1996). This velocity decrease is more significant at low frequencies. For shorter joint lengths the velocity is nearly insensitive to the frequency. Also the attenuation coefficient is affected by the length of the joint. A longer joint will generate a higher attenuation coefficient (Boadu and Long, 1996). At low frequencies (< 200 Hz) the influence of fracture length is negligible, but for higher frequencies the dependence becomes significant. In this case 200 Hz corresponds to a wavelength that is more then 25 times the crack length.

If the contact area increases the amount of reflection will decrease, since the impedance of the fracture increases (Boadu and Long, 1996). For P waves an increase of the angle of incident will decrease the amplitude of the reflected waves. This is also observed for SV waves, although there is an increase in the amplitude of reflected waves when the angle of incident is about 30° to 60°. This is likely to be caused by the increase response to shearing mechanism of the fracture for higher incident angles compared to near vertical angle. The affect of increased contact area is small on SH waves.

Fluids present in a fracture will tend to increase the impedance, especially for P waves. Furthermore, fluids will increase the viscous coupling between the two surfaces. Boadu and Long (1996) investigated the influence of four typical filling material with different viscosities. It was found that an increase in viscosity of the filling material decreased the reflectivity while the transmissibility was increased for both P, SV and SH waves. Increasing the thickness of the filling material (gouge) decreases velocity while the attenuation (damping ratio) increases for S waves (Fratta and Santamarina, 2002). This occurs regardless of stress situation. Thus, the presence of e.g. clay will reduce the wave propagation across a joint.

The angle of incidence influences the amount of transmitted and reflected waves, but also the velocity and the absorption coefficient. Increased angle of incidence increases the velocity of P and SH waves, while the SV velocity wave decreases. The absorption coefficient for P and SH waves is almost insensitive to the angle of incident, while it increases for SV waves with increased angle of incident. The influence of the angle of incidence on the transmission and reflection of waves is too extensive to analyse here and the readers is therefore referred to the work conducted by Boadu and Long (1996).

The frequency of the incident wave influences the transmission and reflection of waves. For P and SV waves, at low frequencies, the amplitude of the reflected waves is small while the amplitude of the transmitted waves is high (Boadu and Long, 1996). This implies that the majority of P and SV waves are transmitted across the fracture, not noticing that there is a fracture. The transmission amplitudes decreases while the reflections amplitudes increase as the frequency increases. The majority of the P waves are reflected for high frequencies. Also the mode conversion increases and at higher frequencies reflected SV are carrying the majority of the energy. SH waves are relatively independent of the frequency of the impinging wave.

Boadu (1996) showed that P wave velocity decreases with decreased Rock Quality Designation value (*RQD*), e.g. a more fractured rock mass will reduce the P wave velocity. It was also observed that there is limit for *RQD* at about 70 % which below the P wave velocity is unchanged regardless of how low *RQD* is. The SH wave velocity also decreases with decreased *RQD* value, but not to the same extent as for P waves. The ratio between P and S waves (c_p/c_s) decreases with decreased *RQD* value.

Fratta and Santamarina (2002) used a relation to distinguish the relation between joint size in a rock mass and wavelength of the wave (λ). The rock mass in divided into block separated by joints, where the block has a thickness, L_r, and a joint aperture, L_j. The salient length scale in the rock mass is then defined as $L_{mass} = L_r + L_j$. This makes it possible to organize wave propagation in jointed rock into two categories; short wavelength (high-frequency) and long-wavelength (low frequency). Short wavelength is when $\lambda << L_{mass}$, and long wavelength is when $\lambda >> L_{mass}$.

3.3.3 Topography

The topography can affect the vibration levels. Nguyen and Catmiri (2007) studied the influence on topography on propagating SV waves with the use of numerical modelling. Three cases where studied; (i) slope, (ii) canyon, and (iii) ridge (shown in Figure 3.5 below). For each case the slope angle and frequencies were altered. Below a summary of the results obtained by Nguyen and Catmiri (2007) is presented.

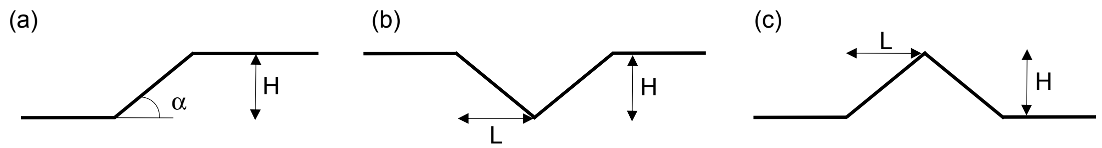


Figure 3.5. The studied topographies: (a) slope, (b) canyon, and (c) ridge (from Nguyen and Catmiri, 2007).

(i) At low frequencies the horizontal movement is increased at the top of the slope while it is attenuated at the bottom of the slope. The position of the minima moves up the slope as the frequency increases, while the position of the maxima remains almost the same. For vertical motions the maximum is observed at the foot while the lowest value is found at the top. For increasing frequencies the maxima is moving towards the upper part of the slope while the minima remains at almost the same position. Increased slope angle amplifies the maxima and minima for both vertical and horizontal movements.

(ii) The movements are low at the bottom of the canyon and increases along the slope. At the top the movements reaches it maximum. For increased frequencies the shatter increases and the maxima and minima are harder to define. Increasing the depth of the canyon amplifies the movements at the crest while the movements at the bottom decreases (are more attenuated).

(iii) The horizontal movement is amplified at the peak of a ridge and attenuated at the foot. Both the amplification and attenuation increases for increased frequency. The minimum is also moving along the slope for increased frequency. The vertical movement is low at the top of the ridge and increase along the slope and reaches its peak at the foot. The position of the maxima and minima are shifted for increased frequencies. Increased height of the rift increases the amplification and attenuation. An increase of the frequency increases the shatter of the movements.

3.3.4 Attenuation in rock masses and soils

As mentioned earlier the amplitude of the propagating waves are attenuated with increasing distance from the source. This is caused by spreading (geometrical damping) and dissipation of energy (material damping). Ungar and Bender (1975) suggested a method to estimate the attenuation (transmission loss) of the waves (vibrations). According to this method the attenuation of P waves, A_T, can be determined from the expression

$$A_T = A_s + A_d + A_i \tag{3.13}$$

where A_s is the spreading loss from a line source, A_d is the attenuation due to internal losses (dissipation) in the soil and rock, and A_i accounts for interfaces between two layers. The terms in Equation 3.13 are presented below.

The attenuation of the amplitude due to spreading from a line source is inversely proportional to the square root of the distance. For an underground tunnel (line-shaped source) Ungar and Bender (1975) calculated the attenuation of the vibration (for P waves) due to spreading as

$$A_s = 10\log_{10}\left(\frac{r_0 + x}{r_0}\right) \tag{3.14}$$

where r_0 is the distance from the centre of the tunnel to the outer wall surface (simply the tunnel radius), and x is the distance from the outer wall surface to an observation point. This would mean that the sound level decreases 3 dB per doubling of distance. The major part of attenuation in rock is caused by geometric spreading (Kurzweil, 1979).

Dissipation (or internal loss) of energy is dependent on the material properties of the medium through which the waves are propagating. Ungar and Bender (1975) estimated the attenuation due to dissipating through the relation

$$A_d = 10 \log e^{-2\pi f x \eta / c} \tag{3.15}$$

where η is the loss factor (see Table 3.1), c is the P wave velocity (see Table 3.1), and f is the frequency of the considered vibration. The dissipation attenuation is frequency dependent. To simplify Ungar and Bender (1975) defined typical properties for three types of soils; see Table 3.1.

Table 3.1 Wave propagation properties for typical soils according to Ungar and Bender (1975).

Soil class	Wave speed[1] [m/s]	Loss factor (η)	Density [kg/m^3]
Rock	3500	0.01	2.65
Sand, silt, gravel, loess	600	0.1	1.6
Clay, clayey soil	1500	0.1-0.2[2]	1.7

1) Longitudinal wave speed.
2) A conservative value since the factor can be as high as 0.5, but such a high value should be used with caution.

When there are changes along the propagation path, usually in the form of interfaces between two different media, attenuation will occur. According to Ungar and Bender (1975) the attenuation at the interface, if the waves impinges normal to the boundary and is moving from layer A to layer C, can be determined from the relation

$$A_i = 20 \log \left[\frac{1}{2} \left(1 + \frac{\rho_C c_C}{\rho_A c_A} \right) \right] \tag{3.16}$$

where ρ is the density and c is the longitudinal wave speed of material A and C. It is assumed that the waves incident normal to the boundary. If $\rho_C c_C < \rho_A c_A$ the attenuation will be negative, which implies that the there is an increase in acceleration levels across the interface. For the above relation to hold the regions of the different media has to be several wavelengths long.

If a layer B is present between layer A and C (for the case where layer A and C are the same), the attenuation provided by the layer, A_l, is determined from (Ungar and Bender, 1975)

$$A_l = 10\log\left[\cos^2\left(\frac{2\pi fl}{c_B}\right) + \frac{1}{4}\left(\frac{\rho_A c_A}{\rho_B c_B} + \frac{\rho_B c_B}{\rho_A c_A}\right)^2 \sin^2\left(\frac{2\pi fl}{c_B}\right)\right] \tag{3.17}$$

where l is the thickness of the layer B, and c_B is the P wave velocity of layer B. From Equation 3.17 is can be observed that the attenuation is highly dependent on the thickness of layer B.

The attenuation of vibration is lower for rock then for soils. Ungar and Bender (1975) determined the vibration levels as a function of distance from a tunnel wall by subtracting the attenuation caused by dissipation and spreading (calculated by adding Eq. 3.14 and Eq. 3.15) from vibration levels measured at tunnel walls in subways. In Figure 3.6 it is clearly shown that the vibration levels are more attenuated when the waves propagates through cohesive soil (sand, silt, gravel, loess) (Figure 3.6b) then when propagating through rock (Figure 3.6a). For clay and clayey soil the attenuation is greater then for cohesive soil (not shown here). It can also be seen that the attenuation of high-frequency vibrations are greater than for low-frequency vibrations when propagating through soil and rock, although the attenuation at higher frequencies are less in rock than in soil.

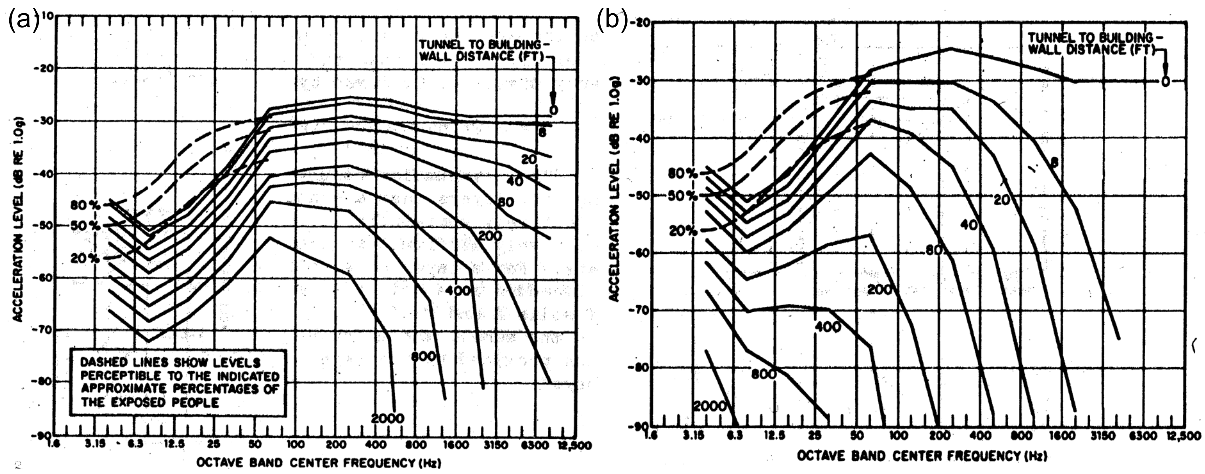


Figure 3.6. Vibration levels in building near subway tunnels as a function of distance from the tunnel, (a) in rock, and (b) in cohesive soil (sand, silt, gravel, loess).

Dobrin and Savit (1988) combined the geometric and material attenuation and the total attenuation for homogenous materials can be determined as

$$A = A_1 \frac{r_1}{r} e^{\alpha(r-r_1)}, \tag{3.18}$$

where A is the amplitude at distance r from the source, A_1 is the amplitude at distance r_1 from the source. The absorption coefficient, α, is determined from

$$\alpha = \frac{\pi f}{Qc}. \tag{3.19}$$

where Q is a quality factor of the material, f is the frequency, and c is the velocity of the propagating wave. The quality factor, Q, is a measure of the damping abilities of the material. The greater the Q value is, the better the rock mass is, i.e. fewer and smaller cracks. Sometime Q^{-1} is used. The quality factor (Q) decreases with decreased *RQD* (Boadu, 1996).

3.4 Analysis methods for vibrations

Methods for analysing the impact on the surrounding from vibration generated by train in tunnels are reviewed within this section. Empirical as well as numerical methods have been included.

3.4.1 Empirical methods

The advantage of empirical formulas is that they are usually simple to use. Since they are more generalized the data, or input, do not need to be accurate. This simplicity is also the main disadvantage for empirical methods. Hence, empirical methods are useable as a first estimate in the preliminary design. Within this section a few simple empirical methods for determining vibration and/or noise levels near train tunnels are described.

Kurzweil (1979) suggested a simple method for estimating the floor vibrations level in a room of a building near a subway. The method takes into account the dynamic properties of both the structure and the soil between the tunnel and the building. This means that changes caused by different design and operation alternatives can be estimated. The floor vibration level, L_a *(room)*, can be estimated from

$$L_a(room) = L_a(tunnel) - C_g - C_{gb} - C_b \tag{3.20}$$

where

L_a *(tunnel)*	=	Octave band acceleration on the wall of a subway tunnel during a train pass by,
C_g	=	the vibration attenuation due to propagation thought the ground
C_{gb}	=	the vibration attenuation (coupling loss) between the ground and the building,
C_b	=	the vibration attenuation due to propagation in the building.

The tunnel wall vibration level, L_a *(tunnel)*, is determined from Figure 3.7 which shows measurements from earth-based (i.e. soil) concrete tunnels. The trains are travelling at about 60 km/h. The lower values are for tracks with smooth rails and wheels, and substructures consisting of both ballasted and direct fixation rail fasteners. It should be noted that there are several parameters that influences the levels of vibration on the tunnel wall; hence adjustment should be made to the value used depending on the condition at the tunnel of interest. For example, for a rock-based tunnel the values in Figure 3.7 can be reduced by 6 dB, and if the thickness of the tunnel wall is doubled it can lead to a reduction in vibrations level of 5 to 18 dB. The reader is referred to Kurzweil (1979) for suggestion to adjustments of the tunnel wall vibration level for various conditions.

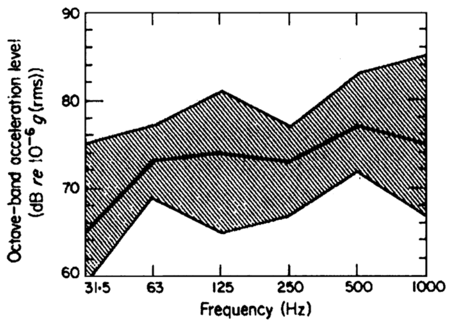


Figure 3.7. Tunnel wall vibration levels. Used to determine L_a (tunnel) in Eq. 3.20 (Kurzweil,1979).

The vibration attenuation due to propagation thought the ground, C_g, is determined from Figure 3.8, which shows the attenuation of ground vibration levels for an average soil as a function of frequency and distance from the tunnel wall. The value (C_g) is thereafter inserted into Eq. 3.20. For rock the attenuation can be determined from

$$C_g(rock) = 10\log_{10}\{(R_0 - X)/R_0\} \tag{3.21}$$

where R_0 is the distance from the tunnel centre to the outer wall surface and X is the distance from that surface to the observation point. This relation is appropriate to use for distances less than one-half of the train length.

The vibration attenuation, or coupling loss between the ground and the building, C_{gb}, is determined from Table 3.2. It can be seen that the coupling loss is dependent on the type of foundation and building type.

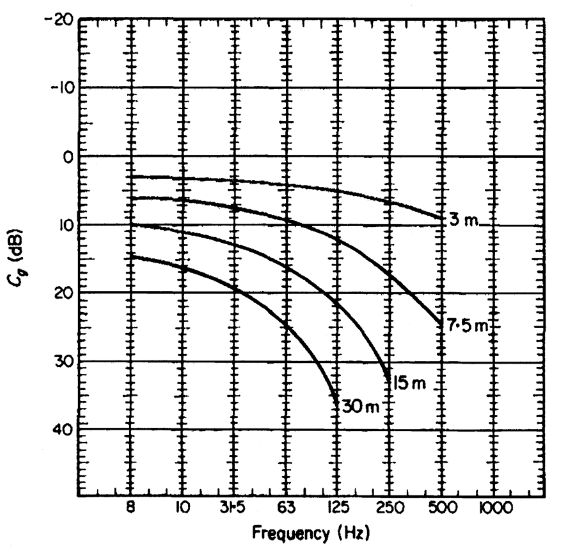


Figure 3.8. Vibration attenuation due to ground propagation. Used to determine C_g in Eq. 3.20 (Kurzweil,1979).

Table 3.2. Coupling loss, C_{gb}, for various foundations and building types (from Kurzweil, 1979).

Foundation and Building type	Coupling loss, C_{gb} [dB]
Lightweight frame building on slab foundation	0
Heavy masonry buildings on spread footings or piles	10 – 20
Footings close to (or attached to) the tunnel structure	0
Footings close to (or attached to) the tunnel structure with resilient material	10 – 20
Building directly on rock	0

The vibration attenuation, or coupling loss, C_b, due to propagation in the building has been set to decrease with 3 dB/floor (starting at the ground level). For lightweight construction the attenuation has been found to be 0, and in some cases it has been found that the vibrations at the upper floors have been amplified due to resonances.

If the sounds pressure level, $L_p\,(room)$, in the room is wanted instead of the acceleration vibration level, $L_a\,(room)$, determined from Eq. 3.20, Kurzweil (1979) used the following relation

$$L_p(room) = L_a(room) - 20\log_{10} f + 37 \qquad (3.22)$$

where f is the octave-band centre frequency.

Melke (1988) suggested a method based on transmission loss to estimate the sound pressure levels in a building. It is assumed that there is a transmission loss along the path between the tunnel construction and the room of interest within the building. The sound pressure levels, or velocity levels, can be estimated from

$$L_B \text{ (dB)} = L_r + R_{tr} + R_{tu} + R_g + R_b \qquad (3.23)$$

where L_r is the rail velocity level, R_{tr} is the track transmission loss, R_{tu} is the tunnel transmission loss, R_g is the ground transmission loss, and R_b is the building transmission loss. This relation is similar to the one suggested by Kurzweil (1979), who also assumed that the sound pressure levels (or vibration levels) present near the rail are attenuated along the propagation path through transmission loss at boundaries and in the ground.

3.4.2 Numerical methods

Since the middle of the 70's the use of computers as a tool to study various problems has increased rapidly, so also within the field of train-induced vibrations. Today it is possible to solve complex vibration transmission problems under realistic conditions, where the computation time mainly depend on the numbers of parameters studied and the accuracy of the grid used within the model.

Depending on the nature of the problem different models can be used. The scale, and whether the discontinuities can be treated separately or not, determines if the rock mass can be treated as a continuum or a discontinuum. Continuum theories can be used when the rock mass is relatively free of joints, or when the discontinuities are closely spaced in comparison with the size of the problem (opening size, path length). The rock mass properties will be chosen so they represent the behaviour of the rock mass (Brown, 1987). When using continuum models the theories of elasticity and plasticity can be used.

For discontinuum theories the displacement fields must not be continuous. It means that individual blocks may be free to rotate, move with slips along discontinuities, or separate at interfaces (Brown, 1987). Discontinuum methods are used when the problem consists of a number of finite, discrete interacting blocks. Common numerical methods used as aid when studying rock mechanic related problems are (Jing, 2003):

- *Continuum* models – the Finite Difference Method (FDM), the Finite Element Method (FEM), and the Boundary Element Method (BEM).

- *Discontinuum* (discrete) models – Discrete Element Method (DEM) and Discrete Fracture Network (DFN) methods.

- *Hybrid* continuum/discontinuum models – Hybrid FEM/BEM, Hybrid DEM/DEM, Hybrid FEM/DEM, and Other hybrid models.

The most common methods used, when studying the propagation of vibration, are the Finite Difference Method (FDM), Finite Element Method (FEM), Boundary Element Method (BEM) and different variants of these methods (Yang and Hsu, 2006). For problems regarding finite domains the FEM and FDM are commonly used, while for problems regarding unbounded domains BEM is the method commonly used. For models based on the FEM or FDM artificial boundaries has to be used if unbounded domains are studied. Furthermore, a BEM model will generate a smaller matrix compared to a FEM model which result in shorter computation times.

Analysis of train-induced vibrations and their propagation can be done with 2D as well as 3D models. The source should be regarded as 3D since the train is a line load, but is often considered (and modelled) as an oscillating force. The propagation path between the source and the receiver is commonly modelled as a plane perpendicular to the tunnel. This is usually sufficient for isotropic ground conditions. However, NGI (2004) and Lai et al. (2005) pointed out that the propagation between source and receiver is a true 3D problem when heterogeneities are included since the vibration propagates along the most beneficial path, which may not be the shortest (i.e. not a plane perpendicular to the tunnel).

Andersen and Jones (2006) compared 2D and 3D numerical models for modelling of the propagation from tunnel to the ground surface. It was found that the 2D model was useful to indicate if reduction or amplification was achieved when there were changes made to the structure (lining and ground conditions), e.g. good for parameter studies. Furthermore, it was recommended that 3D models should be used when absolute vibration transmission predictions were to be determined. However, a major disadvantage of the 3D model was the huge increase in computer time (1000 to 2000 times longer) compared to the 2D model.

Unterberger *et al*. (1997) used *FLAC*, which is a FDM, when performing analysis on train induced vibration on the Vienna metro line. It is a single track railway tunnel situated in clay with a layer of gravel on top. Only the tunnel (lining) and its surrounding ground (soil) was regarded within the model. A harmonic motion was used to induce vibrations. The model was used to determine the influence of groundwater, lining thickness, and tunnel depth on the vibration levels generated at a foundation on the surface. Also the effects of adding a grout body in the invert were studied. It was concluded that numerical analysis are sensitive to the input parameters used and should therefore be regarded as one of several tools when studying vibration related problems.

Lai, et al. (2005) developed a numerical model to analyze train-induced vibrations from the source to the receiver (in this case the human). It was assumed that the vibrations were generated by quasi-static deformation for the axle loads and the dynamic forces from the unevenness of the rails. The influence from sleepers was included along with the lining. Furthermore, it was assumed that there were attenuation as well as damping along the propagation path. A transfer function, based on measurements, was used to determine the propagation from ground to foundation and foundation to the rest of the building. Also the vibration of the floor was included in the model. The model was used to determine the acceleration levels within the building. The results were then compared to the levels recommended by the international standard ISO 2631 (1989).

Degrande, et al. (2006) developed a combined FE and BE model to study the behaviour of a deep bored tunnel and a masonry cut-and-cover tunnel. FEM was used to model the tunnel while BEM was used to model the soil. Only the lining of the tunnel and the surrounding soil was regarded within the model. The model was used to study the response of the tunnels and the ground surface when the tunnel floor was subjected to harmonic loading. The model was also used to determine the in-plane as well as out-of-planed modes.

Fujii, et al. (2005) used FEM to model the vibration in the vicinity of a shielded subway tunnel in Tokyo. Only the tunnel (lining) and ground (soil) is regarded in the model. The load from the train is applied directly on the tunnel floor and was calibrated with the measured vibrations levels near the track. The modelled results were compared to the measured vibration in the ground near the tunnel and on the ground surface. Although the model was simple the results was in good agreement with those measured, especially if the background vibration (from traffic) was regarded.

Andersen and Jones (2006) used coupled FEM and BEM to model a double track cut-and-cover tunnel and a single track tunnel constructed with the New Austrian Tunnelling Method (NATM). The tunnel (lining) was modelled with FEM while the soil was modelled with BEM. The train was modelled as a harmonic motion. Analysis was made in both 2D and 3D. The models were used to determine the vibration level near the tunnel and on the ground surface when the tunnel structure was slightly altered. The aim was to compare 2D with 3D models and thus there were no comparisons with measured vibration levels.

Nagy, et al. (2006) used BEM to determine the noise levels in cuboid rooms within a building. Only the size of the room is regarded within the model. The method was compared to measured noise levels in a room within a building close to an underground railway in Paris. The relation (Eq. 2.2) suggested by Kurzweil (1979) was also used in the comparison. The tendency (trends) of the noise levels agreed very well with the measured and analytical results although there were differences of about 10 dB for some one-third octave bands.

Schillemans (2003) used a 2D model based on FEM to simulate the vibration levels at a tunnel intersection near Antwerp. The model was used to determine what measures that would be necessary to install at critical positions along the railway. To increase the accuracy of the model measured data was used as input.

In the Citytunnel (2000) project FDM was used to model the new tunnel to be constructed in Malmö, Sweden. The program used is called FINDWAVE and is used to model the wave propagation in visco-elastic materials. It is bases on two modules; (i) train module and (ii) track/structure/environment module. The train module is based on numerous train/track parameters (simulated or measured) and is used as input to the model, while the track/structure/environment module models the dynamic behaviour of the track and the structure supporting the train, and the medium surrounding it.

FINDWAVE is used to determine the vibration levels in buildings situated along the planned tunnel for various trains and speeds. It is also used to determine what measure that can be implemented in order to decrease the induced vibrations. The modelled results are compared with values gained from the empirical relation suggested by Ungar and Bender (1975) and measured results from a tunnel in Copenhagen (similar ground conditions).

Table 3.3 presents a compilation of the numerical models used to model train-induced vibrations from tunnels that are mentioned above. All models assume that the ground is homogenous, isotropic, elastic and linear, e.g. the ground is treated as a continuum. Several models do however take into account that the ground is layered, but each layer is homogenous. Another observation is that the majority of tunnels are constructed in soil, which in some cases means clay.

Table 3.3. Compilation of numerical model used for investigation of vibration and/or noise from underground train tunnels.

Model	2D/3D	Real case	Train	Track	Tunnel	Ground	Building	Soil	Rock	Ground assumptions[1]	Reference
FEM+BEM	3D	Yes		x	x	x		x		Layered	Degrande et al. (2006)
BEM	3D	Yes					x			-	Nagy et al. (2006)
FEM	2D	Yes			x	x		x		Layered	Fujii et al. (2005)
FDM	2D	Yes			x	x		x		Layered, Groundwater	Unterberger et al. (1997)
-	2D/3D	Yes[2]	x	(x)	(x)	x	x	x		-	Lai et al. (2005)
FDM	2D	Yes[2]	x	x	x	x	x	x	x	Layered	Citytunneln (2000)
FEM[3]	2D	Yes[2]	(x)	(x)	x	x	x	x		Layered	Schillemans (2003)
FEM+BEM	2D/3D	Yes			x	x		x		-	Andersen and Jones (2006)

What are included in the different categories: Train = Train – rail interaction (can include sleepers); Track = Sleepers, Ballast, Slabs; Tunnel = Lining; Ground = Soil or Rock; Building = Building, or separate elements like Walls, Floor, and Ceiling.

1) If nothing else is stated, the ground is treated as homogenous, isotropic, elastic, and linear.
2) The tunnel is not yet operational; hence no verification between model and measurements has been done yet.
3) In this study several models are used to obtain the complete picture, but the main part (soil) is modeled with the FEM.

4 CODES AND REGULATIONS

4.1 Noise - Acceptable exposure levels

Recommended noise and vibrations levels from Swedish as well as a few international standards are presented within the following sections. The values are recommendations and are based on the noise and vibration levels at which people will be annoyed (disturbed in some way). The majority of the standards for noise and vibration are of general sort and very few consider ground-borne noise or ground-borne vibrations usually associated with trains. There are not as many standards regarding vibrations as there are for noise. To fully understand the standards and their guiding values it is recommended to study each reference in greater depth since most of them contains peculiarities of some kind.

4.1.1 Sweden

Socialstyrelsen (SOSFS, 2005) recommends an equivalent noise level of 30 dB(A) and a maximum level of 45 dB(A) (fast). For low frequency noise (indoor music) an equivalent noise level of 25 dB(A) is recommended and could be used as a guideline values for i.e. train-induced noise. Since noise is frequency dependent Socialstyrelsen (SOSFS, 2005) recommends that different levels for different frequencies should be considered when evaluating noise nuisance (see Table 4.1).

Table 4.1. Recommended equivalent noise levels indoor for various frequencies (SOSFS, 2005).

Frequency [Hz]	31.5	40	50	63	80	100	125	160	200
Noise level [dB]	56	49	43	41.5	40	38	36	34	32

Boverket gives recommended noise levels in dwellings, health-care facilities, and education facilities for noise generated by road traffic (Table 4.2) and installations (Table 4.3) in BFS 1993:57 (1993).

Banverket and Naturvårdsverket (BV, 1997) has, as shown in Table 4.4, determined the maximum allowable noise level indoor buildings near a new, substantially altered, or existing railway. If the stated noise levels are exceeded measures always has to be considered.

Table 4.2. Highest recommended indoor noise levels due to road traffic according to Boverket (BFS 1993:57).

	Equivalent noise level dB(A)	Maximal noise level dB(A) (fast)
Dwellings, daycare centres, and health-care facilities (over one day, i.e. 24 hours)	30	
Sleeping areas in above mentioned facilities during night (22:00 to 06:00)		45*
Workroom intended for office work	40	

*) This level should not be exceeded more than 5 times each night.

Table 4.3. Highest recommended indoor noise levels due to installations according to Boverket (BFS 1993:57).

	Equivalent noise level dB(A)	Maximal noise level dB(A) (fast)**
Dwellings		
- areas for sleep and rest	30*	35
- areas for get-together	30	35
- areas for cooking	35	40
Daycare centres, health-care facilities, schools		
- areas for sleep and rest	30	35
- areas for education	30	35

*) 50 dB(C).

**) This does not include self made noise (from activities).

Table 4.4. Allowable indoor noise levels according to Banverket and Naturvårdsverket (BV 1997).

	Equivalent noise level dB(A)	Maximal noise level dB(A)
Permanent dwellings, leisure dwellings and health-care facilities**	30*	45 (55)***
Educational facilities	30	45
Office	40	60

*) Based on an outside noise of 60 dB(A) and a wall attenuation of 30 dB(A).

**) Value is for bedroom for the time period 22.00-06.00.

***) This value is for buildings along existing railways.

4.1.2 International

In Denmark the Danish Environmental Protection Agency (Miljöstyrelsen) has issued recommendation limits regarding noise in buildings (DEPA, 2002A; 2002B). In the recommendation a special noise levels is set for low frequency noise, i.e. within the frequency range of 10 to 160 Hz (see Table 4.5). The recommended limits for low frequency noise are 5 to 15 dB lower than the ordinary noise levels. For impulse (maximal) noise 5 dB is added to the recommended noise levels.

Table 4.5. Highest recommended indoor noise levels according to the Danish Environmental Protection Agency (DEPA, 2002A).

	Low frequency noise level dB(A)*	Equivalent noise level dB(A)
Dwellings		
- daytime (06:00 – 19:00)	25	30/25
- evening and night (19:00 – 06:00)	20	30
Classroom, office, etc.	30	40
Other room in enterprises	35	50

*) The sound pressure level measured over a reference time period of 10 minutes.

The Federal Transit Administration (a part of U.S. Department of Transportation) has given recommendations for ground-borne noise caused by traffic and trains (Table 4.6). The limiting noise levels are based on the number of events that the building of interest is subjected to. It is believed that people can accept a slightly higher noise level if the events are few. For long and heavy freight trains the frequency of events should be increased to compensate for the long duration of the passage. The special building listed in the lower part of Table 4.6 may have unique specifications for acceptable noise levels and should therefore be determined on a case-by-case basis.

The World Health Organization (WHO) has guidelines and recommended noise levels based on the health effects due to noise (see Table 4.7). Hence, their guiding values are of general character. It is stated in the guidelines that precaution should be taken when there is low-frequency noise present.

Hammarqvist (2004) compiled the recommended noise level limits in dwellings used in train tunnel projects in Sweden, Norway and Denmark. As seen in Table 4.8, the guiding values for the maximal noise level in permanent dwellings are between 30 to 35 dB(A) (maximum; slow) for all projects. The guiding values for the two train tunnels in Copenhagen was

20 to 25 dB(A) (equivalent noise level), where the lowest value was in permanent dwellings during night (Hammarqvist, 2004).

Table 4.6. Highest recommended indoor noise levels according to U.S. Department of Transportation (FTA, 2006).

	Maximal noise level dB(A)		
	Number of events per day		
	Frequent events > 70	Occasional events 70 – 30	Infrequent events < 30
Dwellings where people normally sleep	35	38	43
Institutional land with daytime use	40	43	48
Special buildings	Frequent events > 70	Occasional or Infrequent events < 70	
Concert halls	25	25	
TV Studios	25	25	
Record studios	25	25	
Auditorium	30	38	
Theatres	35	43	

Table 4.7. Highest recommended indoor noise levels according to WHO (WHO, 1999).

	Equivalent noise level dB(A)*	Maximal noise level dB(A) (fast)
Dwellings		
- indoor (daytime)	35	-
- bed room (night time)	30	45
Class rooms, pre-schools		
- indoor (during class)	35	-
- bed room (sleeping time)	30	45
Hospitals		
- ward rooms (daytime)	30	-
- ward rooms (night time)	30	40

*) Time span is 16 hours daytime and 8 hours night time for dwelling and hospital. For schools the time span is during class and during sleeping time.

Table 4.8. Recommended noise levels in dwellings compiled from Scandinavian tunnel projects (from Hammarqvist, 2004).

	Equivalent noise level dB(A)	Maximal noise level dB(A) (slow)
Citytunneln – Malmö	-	30
Citybanan – Stockholm	-	30
Botniabanan – Önsköldsvik	-	30
Train tunnel Verberg – Hamra	-	28 – 34*
Chalmerstunneln – Göteborg (Trolley)	-	30
Train tunnel Falkenberg – Tröingeberg	-	34*
NÄL-tunneln Trollhättan – Öxnered	-	35
Gardemobanan – Oslo	-	30
Sydhamnstunneln – Köpenhamn	30	-
Train tunnel Köpenhamn – Ringsted	20 – 25**	-

*) Modified from fast.

**) Lower value is during night time and higher value is during day time.

In the compilation made by Hammarqvist (2004), where both national and international standards along with recommended levels for a number of train tunnel project in Scandinavia was reviewed, it was concluded that there are some general guidelines that are similar for most countries. The lowest recommended noise levels (25 to 30 dB(A)) are in general stated for buildings that will be used as concert halls (theatres and opera houses) and studios. The noise levels for dwellings and hospitals are accepted to be somewhat higher (30 to 35 dB(A)). Museums, churches, education facilities, etc. should not have higher equivalent noise levels than 35 to 40 dB(A), while office space and other areas where activities are mainly occurring at daytime can expect levels around 40 dB(A). Hammarqvist (2004) concluded that there is a general lack of explanation on how to interpret the recommended noise levels.

4.2 Vibrations - Acceptable exposure levels

4.2.1 Sweden

Banverket (BV, 1997) has, based on the values given by SS 460 48 61, determined the maximum allowable vibration levels in building near a new, substantially altered, or existing railway, see Table 4.9. For a new railway the highest vibration level in a bedroom at night should never exceed 0.4 mm/s, while for substantially altered railways the limit is set to 1.0 mm/s. Banverket has as a long-term goal that no dwelling along existing railways should have a vibration above 1.0 mm/s. The values found in Table 4.9 are well below the limits for structural damage presented in section 2.4.1.

Table 4.9. Allowable vibration levels in a bedroom at night in buildings near a railway (Banverket, 1997).

Vibration level rms (1-80 Hz)	Acceleration [mm/s^2]	Velocity [mm/s]
New railway*	14	0.4
Substantially altered railway*	14	0.4
Existing railways**	36	1.0

*) Refers to permanent dwellings, leisure dwellings and health-care facilities

**) Refers to permanent dwellings

4.2.2 International

The Danish Environmental Protection Agency (Miljöstyrelsen) has issued recommendation regarding vibrations in buildings (see Table 4.10). The guiding values concerns rooms in enterprises (offices) and dwellings in residential and mixed areas, where day and night time are accounted for. The recommended limiting values are given in weighted acceleration level in dB, weighted vibration (mm/s^2) and corresponding weighted velocity (mm/s).

Table 4.10. Allowable vibration levels in buildings according to the Danish Environmental Protection Agency (DEPA, 2002A).

	Weighted acceleration level* [dB]	Weighted acceleration [mm/s^2]	Weighted velocity** [mm/s]
Dwellings in residential areas (day and night) or mixed areas (evening and nights)	75	5.6	0.16
Dwellings in mixed areas (day), offices and classrooms	80	10	0.3
Other rooms in enterprises	85	17.8	0.5

*) Re 10^{-6} m/s^2.

**) Based on the acceleration value.

The Federal Transit Administration (a part of U.S. Department of Transportation) has a criterion for the environmental impact from ground-borne vibrations caused by trains (see Table 4.11). As for noise, the limiting vibration levels are based on the number of events (train passages) that the building of interest is subjected to. The criterion is based on experience from passenger train operation and should therefore be used with caution when applied on freight train. As for noise, the acceptable vibration levels listed in Table 4.11 for special buildings should be determined on a case-by-case basis since the buildings sometimes have unique requirements.

Table 4.11. Highest recommended indoor vibration levels according to U.S. Department of Transportation (FTA, 2006).

	Maximum rms vibration velocity level* [VdB]		
	Number of events per day		
	Frequent events > 70	Occasional events 70 – 30	Infrequent events < 30
Where vibration would interfere with interior operations	65	65	65
Dwellings where people normally sleep	72	75	80
Institutional land with daytime use	75	78	83
Special buildings	Frequent events > 70	Occasional or Infrequent events < 70	
Concert halls	65	65	
TV Studios	65	65	
Record studios	65	65	
Auditorium	72	80	
Theatres	72	80	

*) Re 1 micro-inch/sec.

The limiting values given in Table 4.11 above are used as a first assessment. Once the vibration levels are measured or modelled the gained values can be compared to the limiting values shown in Figure 4.1. The measured or modelled one-third octave band spectra should be plotted together with the curves in Figure 4.1. The frequency octave band (or bands) that exceed the line (criteria) of interest is exceeding the recommended limiting values, and should be mitigated (by appropriate counter measure) to satisfy the recommended criteria. The different criteria and associated vibration level is shown in Table 4.12.

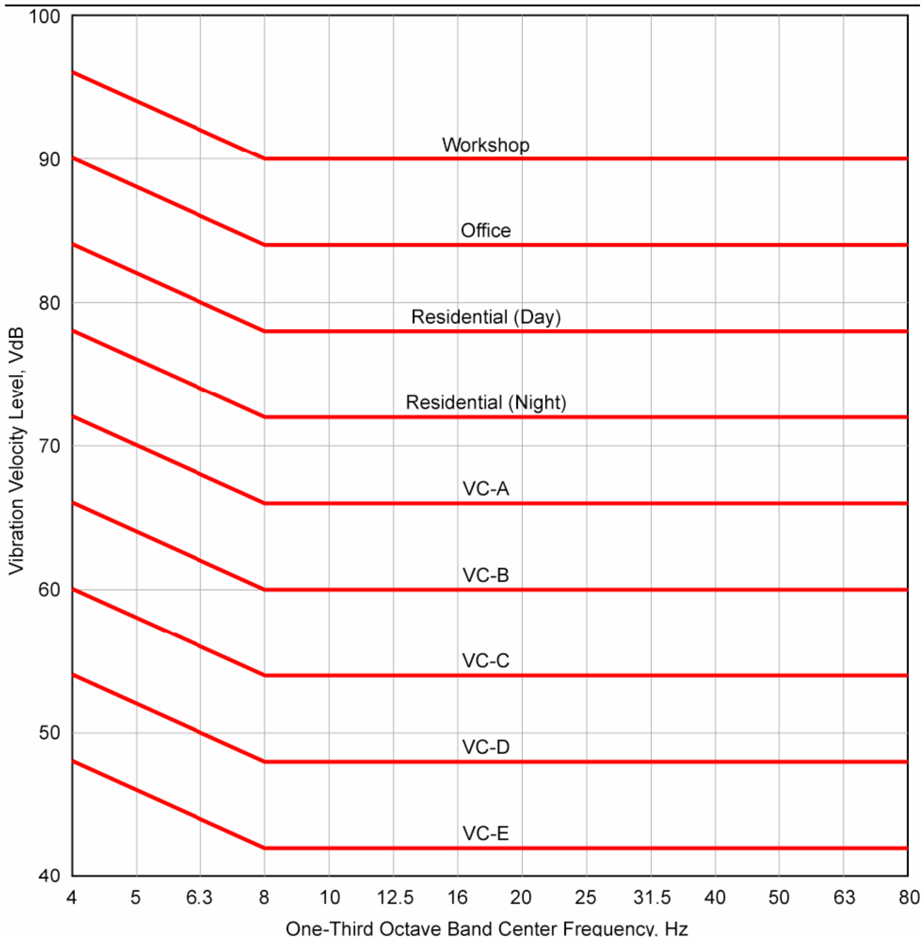


Figure 4.1. Criteria for detailed vibration analysis (FTA, 2006).

The international standardization organization has gathered the magnitudes of building vibrations found to be satisfactory with respect to human response (ISO 2631-2, 1989). These magnitudes can be used as a criterion by multiplying the factors shown in Table 4.13 with the perception base curve shown in Figure 4.2. The resulting curves are shown in Figure 4.2. The combined base curves should be used in the preliminary design to determine if more accurate investigations are needed. ISO 2631-2 (1989) also recommends that single events with high magnitude, such as blasting, should be limited to a small number of events and that they should not take place during night in order to avoid disturbance.

Hammarqvist (2004) has also compiled the recommended vibration level limits in dwellings during the construction phase for a few tunnel projects in Sweden (see Table 4.14). It seems that a vibration velocity of 1.0 mm/s is a common limiting value used in tunnel project in Sweden. It should be noted that there is no limiting value for the Chalmerstunnel in Göteborg. It is assumed that the reason for no limiting value is that the tunnel is completely constructed in rock.

Table 4.12. Interpretation of vibration criteria for detailed analysis (FTA, 2006).

Criterion	Max Lv [VdB]*	Description of use
Workshop	90	Distinctly feelable vibration. Appropriate to workshops and non-sensitive areas.
Office	84	Feelable vibration. Appropriate to offices and non-sensitive areas.
Residential Day	78	Barely feelable vibration. Adequate for computer equipment and low-power optical microscopes (up to 20X).
Residential Night, Operating Rooms	72	Vibration not feelable, but ground-borne noise may be audible inside quiet rooms. Suitable for medium-power optical microscopes (100X) and other equipment of low sensitivity.
VC-A	66	Adequate for medium- to high-power optical microscopes (400X), microbalances, optical balances, and similar specialized equipment.
VC-B	60	Adequate for high-power optical microscopes (1000X), inspection and lithography equipment to 3 micron line widths.
VC-C	54	Appropriate for most lithography and inspection equipment to 1 micron detail size.
VC-D	48	Suitable in most instances for the most demanding equipment, including electron microscopes operating to the limits of their capability.
VC-E	42	The most demanding criterion for extremely vibration-sensitive equipment.

*) Vibration velocity level.

Table 4.13. Multiplying factors used to determine the acceptable vibration magnitudes according to ISO 2631-2 (1989).

Location	Time	Continuous or intermitted	Transient vibrations*
Hospitals operating, theaters laboratories	Day Night	1	1
Residential	Day Night	2 to 4 1.4	30 to 90 1.4 to 20
Office	Day Night	4	60 to 128
Workshop	Day Night	8	90 to 128

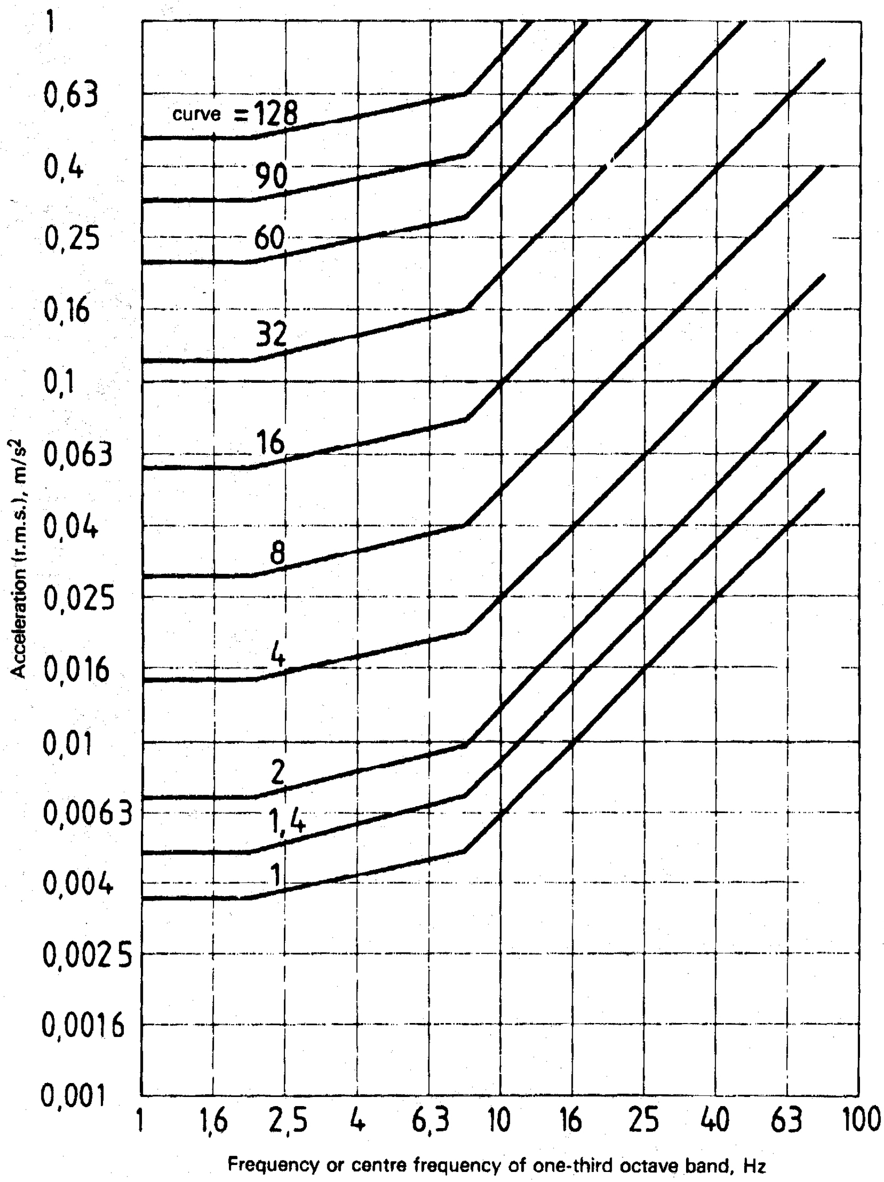


Figure 4.2. Perception base curve for various locations (environments) according to ISO 2631-2 (1989).

The Federal Transit Administration (a part of U.S. Department of Transportation) also has a criterion for building damage due to (construction) vibrations. Limiting values, at which the building is determined to sustain damage, is shown in Table 4.15. The limiting values depend on the material used in the construction. The values presented in Table 4.15 should be used in the early assessment stage to identify possible problem locations that requires a more thoroughly design.

Table 4.14. Recommended noise levels in dwellings due to construction work compiled from Scandinavian tunnel projects (from Hammarqvist, 2004).

Project	Vibration [mm/s]	Comments
Citytunneln – Malmö - Dwellings, hotels and schools - Dwellings mixed areas - Enterprises	 0.4 1.0* 1.0	
Citybanan – Stockholm	1.0	For continuous vibrations. 5 minutes of activity each hour
Norra länken – Stockholm (Traffic)	1.0	The limit was exceeded at some locations and measure were taken
Chalmerstunneln – Göteborg (Trolley)	-	Since it was a rock tunnel no limiting values were used. The noise levels were the limit

*) 1.0 mm/s during day time (07:00 – 18:00), 0.4 during night time (18:00 – 07:00).

Table 4.15. Limiting values for building damage due to vibrations from construction work (FTA, 2006).

Building Category	PPV (in/sec)	Approx. Lv*
Reinforced-concrete, steel or timber (no plaster)	0.5	102
Engineered concrete and masonry (no plaster)	0.3	98
Non-engineered timber and masonry buildings	0.2	94
Buildings extremely susceptible to vibration damage	0.12	90

*) RMS velocity in dB (VdB) re 1 micro-inch/second.

4.3 Standards specifically for train and tunnels

SS-ISO 14837

The International Organization for Standardization (ISO) has compiled a standard for ground-borne noise and vibration from railways. The standard is entitled *Mechanical vibration – Ground-borne noise and vibration arising from rail system* and has the number ISO 14837. The international standard ISO 14837 also works as a national standard for Swedish, labeled SS-ISO 14837 (2005). This standard comprises the following parts:

- Part 1: General guidance (SS-ISO 14837-1:2005)
- Part 2: Prediction models
- Part 3: Measurements
- Part 4: Evaluation criteria
- Part 5: Mitigation
- Part 6: Asset management

Today only *Part 1: General guidance* has been published. The other parts are under preparation, and from the titles it is evident that they will be of interest for research regarding ground-borne noise and ground borne vibrations.

SS-ISO 14837-1 (2005) provides general introduction and guidance for the development of prediction models for situations where ground-borne vibration and/or ground-borne noise may arise in existing or new buildings along an existing or altered railway. The standard defines what essential parts that has to be considered for a model to be determined as reliable. Guidance for calibration and validation of a model is also included within the standard. More detailed guidelines for the different matters will be given in future parts (those listed above and that is in preparation). Criteria and limit values has to be gained from national and international standards, e.g. SS-ISO 2631-1 (1997).

ISO 10815

The international standardization organization has compiled a standard regarding the measurement of vibration in railway tunnels. The standard is entitled *Mechanical vibration – Measurements of vibration generated internally in railway tunnels by the passage of trains* and have the ISO number 10815 (1996). The standard gives suggestion on how and where measurements should be conducted within the tunnel. It also describes what to measure and what equipment that is needed (briefly). See section 5.2.2 for more information.

5 MEASUREMENTS

Measurements are important when trying to understand and explain phenomenon associated with ground-borne vibrations and ground borne noise. Regardless if it is the noise inside an apartment or the vibration levels on the tunnel wall that is to be measured, the same basic principles are applicable. Within the following section some basic concepts are briefly described.

5.1 Noise

5.1.1 Measurements in general

Noise pressure is measured with microphones. The microphone detects the small air pressure variations associated with sound and changes them into electrical signals. If the sound pressure varies harmonically the electric signal will also be harmonic. The relation between the frequency and the size of the microphones is important; if the frequency increases, leading to decreased wavelength, the noise will be influenced. The size of the microphones (e.g. the diameter) is measured in inches where the standardized sizes are; 1”, ½”, ¼”, and 1/8”.

After the microphone has converted the noise to an electrical signal the high impedance is converted to low impedance by a pre-amplifier. Thereafter the signal passes through a low-pass filter which filters the frequency components that are above the range of interest. There exist other filters, such as high-pass filter, band-pass filter, but the low-pass filter is mostly used. The signal is then digitized (converted from analog to digital) and a frequency analysis is performed (e.g. fast Fourier transformation). The data is thereafter manipulated in desired ways (e.g. weighted) and the desired results can be presented, usually in the form of a diagram showing the amplitude as a function of frequency.

5.1.2 Buildings

Ground-borne noise is radiated from vibrating floors, ceilings, and walls. The ground-borne noise levels should be measured at the center of the room to avoid influence from acoustic standing waves in the room (SS-ISO 14837.1, 2005). Ground-borne noise levels measured near the walls are about 2 to 3 dB higher than those measured near the middle of the room. The room should be furnished, unoccupied and with windows closed during the measurements. The frequency range of interest for ground-borne noise within building is 16 to 250 Hz (SS-ISO 14837-1, 2005). General air noise or noise radiating from windows, china, etc. should not be included when measuring ground-borne noise. Ground-borne noise can be derived from ground-borne vibration measurements (SS-ISO 14837.1, 2005).

Ground-borne noise should be quantified using the maximum A-weighted sound pressure level with a slow time constant (L_{pASmax}). The raw un-weighted sound pressure time history should be preserved so that e.g. the equivalent A-weighted sound pressure level (L_{pAeq}) or the one-third-octave band linear spectrum of the event can be derived. When the ground-borne noise is dominated by very low frequencies, the A-weighted sound pressure level may underestimate the subjective response.

5.2 Vibrations

5.2.1 Measurements in general

Vibrations can be measured by several types of transducers. The common ones are (i) acceleration transducers, (ii) displacement transducers, and (iii) velocity transducers. A brief introduction of the transducers is given below.

Acceleration transducer

Acceleration transducers are normally divided into passive and energy conversion, where the later is the most commonly used. Passive means that energy is required for operation and is usually (i) servo accelerometers, (ii) piezoresistive accelerometers, or inductive type accelerometers (eddy current, differential). Active do no require energy to operate and are either (i) seismic accelerometers or (ii) piezoelectric accelerometers. It is most common to measure vibrations with accelerometers.

Velocity transducer

A velocity transducer consists of a coil that moves through a magnetic field (generated by a magnet). Due to the movement, voltage is induced in the coil. The induced voltage is proportional to the relative velocity between the coil and the magnetic field. The coil and magnet is encased within the transducer, where one of them is moving and the other is a part of the transducer frame.

Due to the relatively high voltage output vibration transducers are favorable to use in the field since amplifiers are not required. Velocity transducers are not commonly used since the velocity can be determined by integration of acceleration time histories (acquired by acceleration transducers). However, when measuring very low frequency and low amplitude vibrations (e.g. in buildings and tunnels) velocity transducers are preferred (ISO 2631-1, 1997). One type of velocity transducer is the geophone.

Displacement transducers

A displacement transducer consists of a cased mass suspended by a damper and a spring (a simple single-degree-of-freedom (SDF) system). When the base of the case is subjected to a vibration the mass will move and the displacement can be determined. In order for a displacement transducer to work well the natural frequency of the transducer must be smaller than the measured frequency. Displacement transducers are used to measure floor and ground vibrations, but due to its rather high weight they cannot be used on walls or on light structures.

Instrumentation

The common instrumentation setup for vibration measurements comprises of transducers, and a number of apparatus for amplification, data acquisition and data storage. The instrumentation setup should be capable of measuring vibrations over an appropriate frequency range and magnitudes. The frequency range of interest in a tunnel is from 1 to 500 Hz while the amplitude range is from $5 \cdot 10^{-4}$ to 100 mm/s if the velocity is measured and from $3 \cdot 10^{-6}$ to 500 m/s^2 when acceleration is measured (SS-ISO 14837.1, 2005).

Good coupling with the vibrating medium is required. This can be accomplished by using 300 mm long steel spikes, attachment to building foundations or walls with epoxy resin, double-sided adhesive tape on floors (SS-ISO 24837-1, 2005). For attachment on track components (e.g. rail or sleeper) mechanical fastening is required to accomplish good coupling. It is important to avoid mounted resonance in the frequency range of interest.

5.2.2 Tunnel

The international standardization organization has compiled a standard regarding the measurement of vibration in railway tunnels. It is labeled ISO 10815 (1996). Below the main aspects of vibration measurements in train tunnels is presented (based on the standard ISO 10815). For more guidelines and details the reader is referred to the read the standard.

Vibration measurements in a train tunnel should be conducted where the tunnel is straight over a length of at least 200 m. There should be no visible singular feature (cracks, water, switch point, etc.) near the transducers. The transducers should be oriented in line with the three principal axes of the tunnel (one vertical and two horizontal). According to the international standardization organization vibration tests in tunnels can be conducted in two ways; either as a (i) full test or as a (ii) limited test.

A *full test* requires:

- three measuring sections 20 m apart;
- three pick-ups (either one for each direction or a triaxial unit) at each measure point;
- the train has to make three passages (as a minimum) all in the same direction;
- the signal-to-noise ratio should be higher than 10 dB as overall values and at least 6 dB in each frequency band.

The *limit test* requires:

- only one measure section is needed;
- only the pick-up at one point perpendicular to the relevant plane is needed;
- the signal-to-noise ratio should be equal or higher than 6 dB;
- the train has to make three passages (as a minimum).

For a full or limit test, the track have to be in good condition (no visible flaws or corrugation) and the vehicle should be in a well-maintained condition. The train should be empty and the train composition should be the same as during normal operation. The train should move at the following speeds:

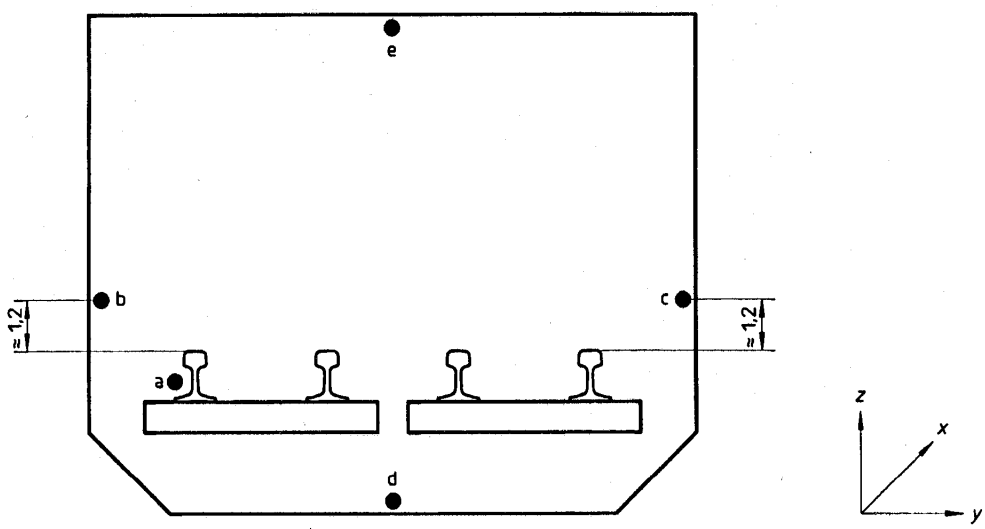
- 40 km/h for tramcars;
- 60 km/h for metropolitan railway trains;
- 80 km/h for rapid transit vehicles;
- the maximum speed allowed at the measure section.

The train should be coasting during the vibration measurements except for the case when the train travels at maximum speed.

The pick-ups should be arranged, for both the full and the limits tests (as shown in Figure 5.1):

- on the invert of the at the cross-section of interest;
- on the vault directly above the pick up on the invert;
- on the tunnel wall, 1.2 m above the level of the rail.

If it is not possible to place a pick-up on the invert, they should be placed between two sleepers if the track is ballasted, or between two successive fasteners or rail spikes on other track types. A pick-up should be placed on the foot of the rail if the relationship between train excitation source and vibration transmitted to the tunnels is of interest. Also the background noise should be recorded so that the signal-to-noise ratio can be determined.



Type of test	Measurement points
Full (see 9.1)	a, b, c, d, e
Limited (see 9.2)	b, d, e

Figure 5.1. Measurements points at a cross-section for full and limited tests (ISO 10815, 1996).

For the measurements on the rail accelerometers should be used, while geophones (velocity transducer) should be used at the other locations. The natural frequency of the pick-ups should be lower than the minimum investigated frequency. The vibration should be measured as a velocity and expressed in mm/s, except the measurement performed on the rail where the acceleration is measured. If decibel scale is used, the reference quantity for vibration is 10^{-6} mm/s while the acceleration reference quantity is 10^{-6} mm/s^2.

5.2.3 Ground

Due to possible variations of the source or the ground conditions measurements should be repeated at a number of locations along the length of the rail system to ensure more reliable results (statistically). For example, transducers could be placed 8, 16, 32, 65 and 125 m from the track at two locations 25 m apart (SS-ISO 14837.1, 2005). It is also required that at least five train pass-bys of each generic category of train (e.g. freight, local commuter, intercity, high speed) is performed. Vibrations levels should be checked at site and if the scatter is greater than ±25 % or ±2 dB a larger number of train pass-bys is needed.

5.2.4 Buildings

Generally, measuring the vertical vibration in the middle of the floor is adequate enough. However, it is preferred that the measurements are conducted in the three orthogonal axes (e.g. two horizontal and one vertical), and preferably parallel and orthogonal to the railway (SS-ISO 14837.1, 2005). If this is not possible, measurements should be performed in the three orthogonal axes of the room or building, while recording the relationship between the railway and room axes. For tall buildings horizontal measurements have to be included, due to horizontal movement of the higher levels. The frequency range of interest within buildings regarding ground-borne vibrations is 1 to 80 Hz (SS-ISO 14837-1, 2005).

The unfiltered time histories should be recorded when conducting vibration measurements inside buildings. This will allow determination of any values when so is desired. It is recommended that building vibrations should be measured as acceleration, but when it is determined to be appropriate vibrations can also be measured in terms of velocity or displacement. ISO 14837-1 (2005) recommends that the overall r.m.s. frequency-weighted acceleration in three orthogonal directions should be used when studying human perception and whole-body response. Some national standards have their own metrics such as peak particle velocity (PPV), vibration dose value (VDV), or KB value (Germany, DIN 4150).

The accelerations signal should, according to ISO 2631-1 (1997), be analyzed and reported as either constant or proportional bandwidth (e.g. one-third octave band). The acceleration signal shall be weighted by factors found in ISO 2631-1 (1997).

6 CASES

This chapter describes a few cases where either measurements or numerical modelling (or both) is conducted to determine ground-borne vibrations or ground-borne noise in buildings or on the surface near a rock or soil tunnel. For each case a short description is given where the use of the tunnel, the measurements and/or modelling performed as well as the results are presented. For more details regarding the cases the reader is referred to the references.

6.1 Citytunneln, Malmö, Sweden

The Citytunnel consists of two 6 km long parallel tunnels under the central regions of Malmö. The tunnels will change Malmö Central Station from a terminus station to a non-terminus station. The tunnels will be connected to the railways north of Malmö which in turn is connected to the railway from Copenhagen. About 4.5 km of the Citytunnel is excavated with the use of tunnel boring machines (TMB) while 1.5 is excavated from the surface (open cuts) and thereafter covered. The diameter of the tunnels is 7.9 m. Escape tunnels (cross tunnels) are constructed between the two tunnel tubes every 350 m.

Results

The aim of the study was to predict the ground-borne noise levels generated from the operation of the Citytunnel (Citytunneln, 2000). Numerical simulations were carried out for ten selected buildings presumed to be sensitive to vibration (a hospital, a school, a theatre and old historical buildings). The software used is called FINDWAVE and is based on FDM (finite difference method). The model used was validated by comparing modeled transmission loss with results from measurements conducted at the metro tunnel in Copenhagen (similar rock type: limestone). See section 6.2 for more information regarding the Copenhagen Metro.

The ground was modeled as a layered medium, where the input data was taken from the geological investigation conducted within the Malmö area (boreholes). The modelled train speeds for the X2000 train were 80 and 120 km/h. The result from the modelling showed that the ground-borne noise levels within many of the buildings would be in the region of 46 to 55 dB(A), which is significantly higher than the levels considered as acceptable according to most of the standards. If resilient baseplates are used instead of ballasted track the ground-borne noise levels will decrease and be in the region of 33 to 43 dB(A). It was mentioned that the condition of the rail was of great importance, and if the rail was in better or worse condition than the simulated rails, experience had shown that the ground-borne noise level could be ±10 dB the modelled levels. Another great source to uncertaintys was the conditions of the ground, such as damping characteristics but also the predicted loss factor for the Limestone.

The results from the numerical modeling cannot be verified yet since the Citytunnel is still under construction and thus no vibration measurements from train traffic have yet been conducted.

6.2 Metro tunnel in Copenhagen, Denmark

The Copenhagen Metro is the first ever metro system in Denmark. Once it is completed, the metro will cover 21 km of which 10 km is below surface. Most of the underground parts are within the central parts of Copenhagen. The tunnel part consists of two tunnel tubes running side by side along the route. Most of the tunnel is constructed using TBM (tunnel boring machine), but areas where is has been necessary has been excavated in accordance to NATM (new Austrian tunneling method) or by conventional cut-and-cover tunneling.

In Copenhagen the upper layers consist of moraine deposits on top of limestone. In the central part the limestone consist of hard flint layers while the outer parts consists of clayed sandy moraine (see Figure 6.1). The diameter of the bored tunnel is 5.5 m, and when the concrete panels are added the inner diameter is reduced to 4.9 m. The rails are mounted on an even concrete surface where the sleeper and other potential obstacles are not visible.

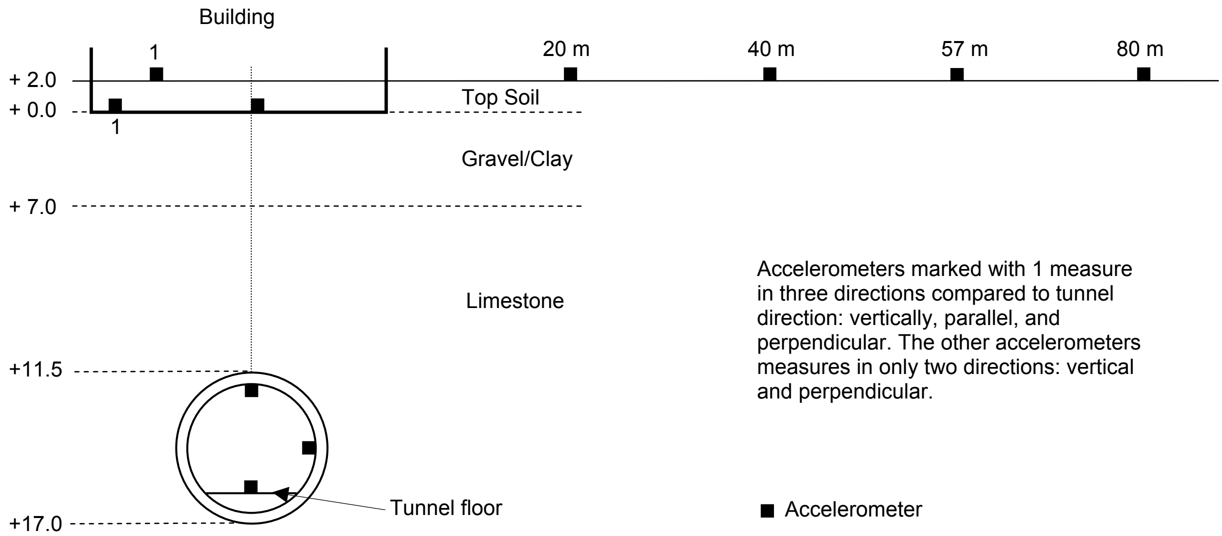


Figure 6.1. Cross section of the metro tunnel in Copenhagen showing the placement of the accelerometers and the ground conditions (modified from Citytunneln, 2000).

Measurements

Measurements has been conducted to determine the transmission loss between the tunnel floor and (i) the tunnel wall, (ii) the tunnel ceiling, (iii) floors in a building above the tunnel (in different rooms and on the ground outside of the building), and (iv) the ground surface (Citytunneln, 2000). The vibration levels were measured with accelerometers placed at each measurement point (as seen in Figure 6.1). Both vertical and horizontal vibrations were measured at each measurement point. Below the transmission loss for the different test is presented. (There is scatter in the results but the general trends are prominent.)

(i) The transmission loss between the tunnel floor and the tunnel wall is relatively constant 25 dB up to 25 Hz after which it starts to increase up to about 40 dB at 63 Hz (the results are scattered above 63 Hz).

(ii) The transmission loss between the tunnel floor and the tunnel ceiling is relatively constant 20 dB up to 125 Hz after which it starts to decrease.

(iii) The transmission loss between tunnel floor and the floors in the building is increasing as the frequency increases for both vertical and horizontal vibrations (25 to 30 dB below 20 Hz and rises to 50 ot 70 at 500 Hz). The transmission loss in the horizontal direction is slightly higher than the transmission loss in the vertical direction (about 10 dB higher in general). The transmission loss between tunnel floor and the ground surface just outside of the building is increasing with increased frequency, just as was observed for the floors inside the building. However, the transmission loss in the horizontal direction is equal (or slightly lower) to the transmission loss in the vertical direction.

(iv) Vertical and horizontal vibration levels are decreasing with increasing distance between tunnel floor and measurement point on the ground surface. The transmission loss is frequency dependent; high losses at high frequencies and low losses for low frequencies (16 to 32 Hz). The decrease as a function of distance is smaller in the horizontal direction than in the vertical direction.

Furthermore, it was observed the tunnel floor moved downward while the walls moved inward towards the tunnel centre, at lower frequencies. This means that the tunnel is stretched in the vertical direction when the floor is subjected to a force.

Results

The transmission loss between tunnel floor and the floors in the building and outside the building was modeled with FINDWAVE. It was found that the model sometime overpredicted and sometimes underpredicted the transmission loss, both in the vertical and horizontal directions. The prediction error was found to be greater for the building (rooms) than for the position just outside of the building. The prediction error could be up to 20 to 30 dB at some frequencies, and was in general larger in the horizontal direction. (A prediction error of 20 to 30 dB is at some frequencies half of the measured transmission loss.) It was concluded that the model predicted the transmission loss quiet well within the 50 to 100 Hz frequency range (something that definitely can be discussed).

6.3 Gårdatunneln, Göteborg, Sweden

The Gårdatunnel is a 2163 m long two track single tube tunnel in central Göteborg that is used by south bound trains (destination Malmö). Both intercity trains and freight train passes throught the tunnel. The tunnel is constructed in rock with varying overburden. There are apartment buildings as well as family dwellings above the tunnel

Before the measurements were conducted a questionaire was sent out to 200 of the households living above or near the tunnel (WSP, 2005). 30 of the households replied on the questionaire, and from them six objects were selected. The questionaire concerned how the residents experienced noise and vibrations but also how they in general were affected by the train traffic in the tunnel.

Measurements

Vibration measurements where conducted during evening time in three apartments and three private homes (WSP, 2005). Both intercity trains and freight train passages were measured. The ground-borne vibrations were measured with a triaxial geophone (three orthogonal directions) placed on the floor. The measured frequency range was 2 to 80 Hz (slow). The ground-borne noise was measured with a microphone within the frequency range 12.5 to 5000 Hz (terser band, fast and slow).

More detailed measurements were conducted in one apartment and in one private home, where pronounced ground-borne noise was observed. Microphones were placed in three independent positions (where one was in a corner), while an accelerometer was placed on the floor (vertically). The noise and vibrations measurements were conducted simultaneously.

Results

The equivalent and maximal sound pressure levels were measured within the six buildings. The equivalent sound pressure levels were compared to SOSFS 1996:7 recommended terser band values (the same values as given in SOSFS 2005:6 (2005)). When freight trains passed through the tunnel, the equivalent sound pressure levels were found to exceed the recommended values at frequencies above 50 Hz for two buildings, while at 125 Hz the recommended values were exceeded in a majority of the buildings (5 out of 6). Freight trains generated higher noise pressure levels and had a longer duration than inter city trains.

The maximal sound pressure levels were compared to the hearing threshold levels given by ISO 1975. When freight trains passed through the tunnel, the maximal sound pressure levels exceeded the hearing threshold for frequencies above 125 Hz in one building, while the maximal sound pressure levels exceeded the hearing threshold at 160 Hz in all six buildings. The ground-borne noise from freight train passages was hearable in all six buildings. In some building the maximal noise pressure levels were above 40 dBA.

Some of the vibration measurements failed due to problems with the equipment. Based on the limited measurements of freight train passages it was determined that the vibration levels were below 0.1 m/s^2, which is below the comfort value of 0.4 m/s^2. It was concluded that the vibration was too small and therefore did not contribute to an increased noise pressure levels. The vibrations reached the building (was measurable) 5 to 10 seconds before the ground-borne noise was detectable within the room.

It was concluded that in the future, with increased train speed, increased axle loads and increased frequency of trains, there is an increased risk of problems due to ground-borne noise. It was therefore suggested that railways in tunnels beneath communities should be constructed with the best possible technology to reduce the presence of ground-borne noise from pass by trains.

6.4 Double Tracked Line Sandvika – Asker (Askerbanen), Norway

The double tacked line tunnel near the Asker train station is part of the restoration (renovation as well as construction of new of stretches) of the railway between Sandvika and Asker (Askerbanen). The railway tunnel is running below several dwellings (private houses). The overburden consists of rock and soil and is only a few meters thick.

Analysis

A computer program was developed in the program MATLAB, which could simulate how various track structures transmit vibrations from the track down to the rock (tunnel floor) (NGI 2004; Cleave et al., 2005A, Cleave et al., 2005B). The numerical model simulates one dimensional wave transmission in a vertical cross-section of the track. Within the model, paramters such as differences in material properties between horizontal layers, dynamic properties of the rolling stock, and counter measures (e.g. rails pads or mats) are considered. The results from the numerical model are presented in the frequency domain.

Measurments

A full scale test was conducted near the entrance of the tunnel (NGI 2004; Cleave et al., 2005B). Seven test sites with different track structures were established (based on the results from the numerical simulation in MATLAB), each site being 8 m long. A converted two axle freight wagon was used as load, where servo hydraulic actuator on one of the wheel axles was used as the dynamic load. The load on each axle was 225 kN from the loaded freight wagon. The (dynamic) load capacity of the actuator was ±10 kN and could be generated at frequencies between 20 to 350 Hz.

The vibrations were measured by accelerometers at several different locations within the track structure, e.g. on the sleeper, in the ballast, at the ballast mat, in the rock fill, etc. Accelerometers were also placed on the rail, on the tunnel floor (below the track structure), and on the tunnel wall (both sides). The accelerometers were fixed to aluminum plates when located within the track structure and on the tunnel floor (where the plates where cast into concrete). On the wall the accelerometers were attached to preinstalled rock bolts. The force of the actuator and the dynamic power between the wheel and rail was also measured. The recorded data was analyzed with MATLAB and the results were presented in one-third octave bands.

The main objective of the study was to quantify the reduction of the structure borne noise transmission for different track structures. The reduction was measured in form of insertion loss which is the difference between one tested track structure compared to a reference or base structure. The insertion loss is based on the transfer of vibration energy from the rail to the tunnel floor, tunnel wall or the floor in the residential houses above or nearby the tunnel.

Results

The standard track structure was used as reference. Such a track structure consist of; the tunnel floor (leveled by well graded compacted blasted rock), a 100 mm layer of crushed rock (4 to 32 mm), a 600 mm layer of ballast, and concrete sleeper with rail pads. For the six tested track structures different properties were altered, such as; the addition of a 1.6 m layer of blasted rock, insertion of ballast mats, or the use of a 0.8 m layer of LECA together with 0.8 m of blasted rock. It was concluded that all altered track structures had a reducing effect on the structure borne noise at the tunnel floor (up to 40 dB) and at the tunnel wall (up to 30 dB). There were differences between the insertion losses determined at the tunnel floor and those on the wall. It was determined that the variation of the rock properties between the different test sites was considerable and hence overshadowed the difference between the various track structures.

Measurements were also conducted in four residences above the tunnel. The vibration was measured with an accelerometer placed on the lowest floor while the noise was measured with a microphone (in the same room as the accelerometer). Since each test site is 8 m long, the distance between the excitation point (wagon load) and the measure point will vary (at most about 23 m). As was observed for the tunnel floor and wall, all tested track structures performed better than the standard track structure. However, the measurements gave inconclusive results since the variation of the transmitting properties of the bedrock between each site was larger than the variations between the different between the track structures. Thus, it was concluded that it is the section (or path) that best transmits the vibrations that will cause the highest noise levels in nearby residences.

6.5 Double track tunnel in Tokyo, Japan

The tunnel is a double track tunnel which 8-car subway trains passes through at a speed of 45 km/s. The tunnel is circular with an outer diameter of 10.4 m and a 0.8 m thick concrete lining (segments). There were no buildings above the tunnel and the ground surface was flat.

At the section of measurements the overburden was 15.8 m. The ground consists of sandy or clayey soil in the upper layers, and diluvium sand at the level at which the tunnel were located. The shear velocity, Poisson's ratio, density and other factors were determined for the different layers by boring explorations or PS logging. Vibration acceleration was measured on and under the ground surface, as well as inside the tunnel (Fujii et al., 2005). On the surface accelerometers were located right above the tunnel, and at various distances along a line perpendicular to the tunnel (at distances from 5.2 m up to 40.2 m). Accelerometers were placed underground at different depth and different distances from the tunnel (7.2 m from the tunnel centerline at and 5, 15 and 25 m depth). Inside the tunnel accelerometers where located on the floor and wall.

The measured vibration accelerations were low at low frequencies (< 16 Hz). The levels then increased and leveled out at 63 Hz (as seen in Figure 2.3). The vibration at the track had the peak at 315 Hz while the vibrations on the wall started do decrease at about 200 Hz. The vibration acceleration level parallel to the tunnel was 10 dB less than the vertical and horizontal (perpendicular to the tunnel) vibrations. The vibration on the surface reaches a maximum (about 10 dB lower that in the tunnel) at about 50 Hz and is then attenuated in similar manner for all distances from the tunnel centerline. Within the ground, the vibrations at the accelerometer at 25 m depth (almost in level with the tunnel bottom) are only slightly attenuated at frequencies above 250 Hz. When moving towards the surface (15 and 5 m depth) the attenuation increases and becomes similar to those for vibration measured on the ground surface.

The vibration propagation measured within the section was analyzed by a 2D FEM (Finite Element Method) model (Fujii et al., 2005). The soil was modeled as solid elements while the tunnel was modeled as beam elements. The upper layers (sandy and clayey soil) were modeled with a fine mesh size (0.2 m x 0.4 m) while the layers near the tunnel were modeled with a bigger mesh size (0.4 m x 0.4 m). Properties of the track and train were not included since the exciting force (based on the measured levels near the track) was added at the track center position.

The results from the numerical model were compared with the results from the measurements. It was determined that their agreement were adequate. At low frequencies there were differences, but if the background noise (from traffic on the ground surface) was added to the model, good agreement between modeled and measured results was obtained. It was concluded that if the vibration acceleration levels near the track in a tunnel is known, the vibration levels on the ground surface can be predicted.

6.6 Summary

The above cases contains varying amount of information concerning the ground conditions, measuring setup, tunnel specifications, building conditions, etc. Nevertheless, the do offer the possibility to verify the results from numerical models with results from measurements. Knowing that a basic model is producing reliable results is valuable once more complex models are to be performed, e.g. adding faults, varying topography, etc. into the model.

7 DISCUSSION AND CONCLUSIONS

7.1 Discussion

Within this report the state of the art of vibrations induced by passing trains in tunnels has been reviewed. Propagation of train-induced vibrations is usually divided into three parts;

- *Source* – includes train, rail, velocity, train type, sleepers, etc.
- *(Propagation) Path* – includes embankment, tunnel wall (lining), rock, soil, etc.
- *Receiver* – includes foundation, building, people, etc.

The emphasis has been on the propagation path which in this case is the rock mass. To cover the whole system the source, e.g. the train-rail interaction, and the receiver, e.g. buildings and human's response has been reviewed as well although only briefly.

The properties and the behaviour of many factors involved in the generation and propagation of the vibration has to be known to carry out a relevant analysis of how vibrations generated by trains affect buildings and people within the buildings. This area spans over several research disciplines such as physics, solid mechanics, geophysics, soil and rock mechanics, structural dynamics and the human response (physiological as well as psychological). Hence, in order to be able to study train induced vibrations simplifications have to be done. The choice of simplifications and assumptions depends on the background of the person carrying out the analysis.

Today, there is no general relation that accounts for all phenomena occurring between the source and the receiver. There are methods that consider many of the above listed factors, but usually simplifications and assumptions, sometimes rough, are made to make it possible to carry out any analysis. This gives relations that are easy to use but may provide inaccurate results. Furthermore, due to the simplifications and assumptions some relations can only be used in certain specific cases (one-dimensional, only soil, etc.)

The effect of vibrations on buildings and humans (receivers) is well known. The same holds for the generation of the vibration as a result of the interaction between rail and wheel for open tracks on soil. Consequently, the reader is referred to the reference list for further discussion about matters concerning these subjects. This study was initiated to review the state of the art regarding the propagation of waves (vibrations) from the track to the ground surface, and has therefore focused on how the rock mass and the soil affect the propagation.

For the problem of structure-borne vibration caused by moving trains in tunnels the understanding of wave propagation through the ground is of great importance. The main interest is to determine whether the waves are attenuated or amplified, and if the frequency content is changes in an unfavourable way. Attenuation is caused by geometrical spreading, material damping (friction) and reflections and refractions.

In an isotropic, homogenous, and linear elastic material with infinite extent the behaviour of the stress waves are determined from closed form relations that are rather easy to use and understand. Adding a free surface (half space) or a surface between two solids (boundary), causes reflection and refraction of waves and increases the complexity of the problem slightly. Still theoretical relations can be used to study local phenomenon.

In a rock mass heterogeneities of various sizes are present and the material can not be classified as an ideal material, and therefore material damping also has to be accounted for. Hence, wave propagation in a rock mass is influenced by properties such as

- *Discontinuities* – joints, faults, cracks, crushed zones, dykes, etc. Important properties: length, opening, width, filling material, inclination (to the wave), amount, etc.
- *Boundaries* – between different rock types or soil. Depends on differences in e.g. density, velocity, stiffness, soil, clay, etc.
- *Topography* – along as well as crossing tunnels. Depends on e.g. slope angle, height (ridge) or depth (canyon).

Higher amount of joints, faults and boundaries increases the attenuation of the waves. Therefore, the rock mass can be seen as a filter where the high-frequency content is filtered since higher frequencies are more influenced by the above listed properties. These properties may also cause amplification if the conditions are favourable. Therefore, to be able to analyze the effect of the geomaterials with analytical methods extensive assumptions have to be done. Hence, numerical analysis is therefore the only option. This also allows for adding faults, water conditions, topography, and variations in stiffness, etc., into the model. However, still an understanding of the phenomenon that can occur when waves are propagating through the rock mass is necessary in order to evaluate the results from numerical analyses.

There are numerical models and tools available to study the propagation of train-induced vibrations from source to receiver. The common approach is to study a cross-section of a tunnel (i.e. 2D), where the surrounding ground is included. 2D models allow the creation of simple models to study the influence of individual factors/parameters, but also the creation of more advanced models including several factors at once. However, for complicated geometries, e.g. joints or faults intersecting the tunnel or if the tunnel is passing through both rock and soil due to variation in topography, 3D models will probably be necessary.

A whole variety of models for analyses of train induced vibrations is reported in the literature. The difference among the models depends mainly on which phenomenon the authors have chosen to study. This applies to both 2D and 3D models. Normally, the focus is on the source and hence many factors concerning the train-wheel interaction is included in these models. Sometimes various countermeasures, e.g. floating slabs, are included, or the building (often only as a frame). Common for all numerical models is that the rock mass, or soil, is assumed to be homogenous and isotropic. It should be noted that a common explanation to observed differences in the results between numerical models and measurements is the presence of various heterogeneities within the ground (e.g. Lai et al, 2005). However, their true effect is never investigated.

7.2 Conclusions

From this literature review the state of the art on the prediction of vibrations on the surface or in nearby buildings from trains moving in tunnels can be summarized as follows:

- Analyses of vibrations induced by trains in tunnels, the rock or soil, are always simulated as isotropic, homogenous and elastic material. This means that the influence of joint, faults, boundaries, ground water level, adjacent tunnels, topography, etc. is not included in the models.

- A large quantity of the research focusing on wave propagation through rock and soil has been published. They represent for instance, laboratory tests on small scale samples as well as research from the field of seismology and blasting. However, there are very few publications dealing with train (in tunnels) induced wave propagation through geomaterials, including the complexity associated with it.

- Empirical methods are inaccurate due to many simplification and assumptions. They do generally not consider the properties of the ground at any great detail and are site specific.

There are several different numerical methods and solutions available to analyze vibrations from trains in tunnels and their impact on the surroundings. However, many of them are developed for specific cases (soil, dry conditions, etc), or are focused on one aspect of the source-path-receiver path (i.e. rail-wheel interaction, rail-sleeper interaction, damping effects of countermeasures below the rail, etc.).

7.3 Recommendations for future work

This survey has shown that heterogeneities present within the rock mass (or soil) are disregarded when studying the propagation of vibration from trains in tunnels. It has also been shown that there is a lack of understanding on how waves with a frequency content typical for train induced vibrations propagate through ground with various heterogeneities.

It is therefore recommended to investigate

- the propagation of waves within a rock mass containing different heterogeneities using conceptual models without any tunnel, train or building.

- the propagation of waves through the rock mass including different heterogeneities using conceptual models. These models should include the tunnel, the embankment, and the ground surface. The ground should include the cases of a ground consisting of only rock as well as rock covered by soil.

- a number of real cases with well documented vibrations measurements and rock mass descriptions.

The models should be of analytical, empirical and numerical nature, and should be performed in both 1D and 2D. Measured data from a few cases where the geological properties are well known should be used together with the numerical model for validation purposes.

If possible, the use of conceptual models together with measurements from a few cases might result in classification guidelines usable for train tunnels constructed in rock masses.

8 REFERENCES

Achenbach, J.D. (1973). *Wave propagation in elastic solids*. Third printing. Amsterdam: North-Holland Publishing Company.

Adam, M. and von Estorff, O. (2005). Reduction of train-induced building vibrations by using open and filled trenches. *Computer & structures*, **83**, 11–24.

Amadei, B. (1996). Importance of anisotropy when estimating and measuring in situ stresses in rock. *International Journal of Rock Mechanics and Mining Science & Geome-chanical Abstract*, **33 (3)**, 293-325.

Andersena, L. and Jones, C.J.C. (2006). Coupled boundary and finite element analysis of vibration from railway tunnels—a comparison of two- and three-dimensional models. *Journal of Sound and Vibration*, **293**, 611–625.

BFS 1993:57. (1993). Boverkets författningssamling (BBR 94). (In Swedish).

Boadu, F.K. (1997). Fractured rock mass characterization parameters and seismic properties: Analytical studies. *Journal of Applied Geophysics*, **37(1)**, 1-19.

Boadu, F.K. and Long, L.T. (1996). Effects of fractures on seismic-wave velocity and attenuation. *Geophysical Journal International*, **127(1)**, 86-110.

Brown, E.T. (1987). Introduction. In: *Analytical and computational methods in engineering rock mechanics*. Ed. by Brown, E.T. London: Allen & Unwin.

BV (1997). *Buller och vibrationer från spårburen linjetrafik. Riktlinjer och tillämpning*. BVPO 724.001. Banverket och Naturvårdsverket. (In Swedish).

Bodén, H., Carlsson, U., Glav, R., Wallin, H.P. and Åbom, M. (2001). *Ljud och vibrationer*. 2nd edition. Stockholm: Nordstedt. (In Swedish).

Citytunnel (2000). *Noise and vibration study project. Prediction model for Citytunneln. Final report, part 2 detailed*. AB 60 MD 560 0001. Citytunnelkonsortiet.

Cleave, R., Madshus, C., Grande, L., Brekke, A. and Rothshild, K. (2005A). Mitigation of ground borne noise in rock railway tunnels – Part I: Track design and simulation. In *7th*

International Conference on the Bearing Capacity of Roads, Railways and Airfields, Trondheim, June 27-29 2005.

Cleave, R., Madshus, C., Grande, L., Brekke, A. and Rothshild, K. (2005B). Mitigation of ground borne noise in rock railway tunnels – Part II: Full scale tests. In *7th International Conference on the Bearing Capacity of Roads, Railways and Airfields, Trondheim, June 27-29 2005.*

Das, B.M. (1993). *Principles of soil dynamics*. Boston: PWS-KENT Publishing Company.

Davis, M.L. and Cornwell, D.A. (1998). *Introduction to environmental engineering*. 3rd edition. WCB/McGraw-Hill.

Dawn, T.M. and Stanworth, C.G. (1979). Ground vibrations from passing trains. *Journal of Sound and Vibration*, **66(3)**, 355–362.

Degrande, G., Clouteau, D., Othman, R., Arnst, M., Chebli, H., Klein, R., Chatterjee, P. and Janssens, B. (2006). A numerical model for ground-borne vibrations from underground railway traffic based on a periodic finite element–boundary element formulation. *Journal of Sound and Vibration*, **293**, 645–666.

Deischl, F., Eisenmann, L. and Steinbeisser, L. (1995). Railways. In: *Vibration problems in structures*. Ed. by Bachman, H. Basel: Birkäuser Verlag.

DEPA. (2002A). *Danish guidelines on environmental low frequency noise, infrasound and vibration.* Danish Environmental Protection Agency. http://glwww.mst.dk/transportuk/02030000.htm. (2007-11-19).

DEPA. (2002B). *Recommended noise limits.* Danish Environmental Protection Agency. http://glwww.mst.dk/transportuk/02050000.htm. (2007-11-19).

Dobrin, M.B. and Savit, C.H. (1988). *Introduction to geophysical prospecting*. 4th edition. McGraw-Hill, Inc.

Dowding, C.H. (1996). *Construction vibration*. Prentice-Hall.

Ford, R.D. (1987). Physical assessment of transportation noise. In: *Transportation noise reference book*. Ed. by Nelson. P.M. London: Butterworths.

Fratta, D. and Santamarina, J.C. (2002). Shear wave propagation in jointed rock: state of stress. *Géotechnique*, **52(7)**, 495–505.

FTA (2006). *Transit noise and vibration impact assessment.* Office of Planning and Environment, Federal Transit Administration. Report number: FTA-VA-90-1003-06.

Fujii, K., Takei, Y. and Tsuno, K. (2005). Propagation properties of train-induced vibrations from tunnels. *Quarterly Report of RTRI (Railway Technical Research Institute) (Japan)*, **46(3)**, 194-199.

Griffin. M.J. (1990). *Handbook of human vibrations*. San Diego: Academic Press.

Hall, L. (2003). Simulations and analyses of train-induced ground vibrations in finite element models. *Soil Dynamics and Earthquake Engineering*, **23**, 403–413.

Hammarqvist, M. (2004). *Järnvägsutredning västlänken. Buller & vibrationer, inventering.* PM56. Banverket. (In Swedish).

Hemsworth, B. (2000). Reducing groundborne vibrations: state-of-the-art study. *Journal of Sound and Vibration*, **231(3)**, 703–709.

Howarth, H.V.C. and Griffin, M.J. (1990). Subjective response to combined noise and vibration: Summation and interaction effects. *Journal of Sound and Vibration*, **143(3)**, 443–454.

Hung, H.H., Yang, Y.B., Asce, F. and Chang, W. (2004). Wave barrier for reduction of train-induced vibrations in soils. *Journal of Geotechnical and Geoenvironmental Engineering,* **130**, 1283–1291.

Hunt, H.E.M. (2001). Measures for reducing ground vibration generated by trains in tunnels. In: *Noise and vibration from high-speed trains*. Ed. by Krylov, V.V. 423-430. London: Thomas Telford.

ISO 10815 (1996). *Mechanical vibration – Measurements of vibration generated internally in railway tunnels by the passage of trains.*

ISO 2631-1 (1997). *Mechanical vibration and shock – Evaluation of human exposure to whole-body vibration – Part 1: General requirements.*

ISO 2631-2 (1989). *Mechanical vibration and shock – Evaluation of human exposure to whole-body vibration – Part 2: Continuous and shock-induced vibration in buildings (1 to 80 Hz).*

Jing, L. (2003). A review of techniques, advances and outstanding issues in numerical modelling for rock mechanics and rock engineering. *International Journal of Rock Mechanics and Mining Science*, **40**, 283-353.

Kazamaki. T. and Watanabe, T. (1975). Reduction of solid borne sound from a subway. In: *Proceeding of the 4th International Conferences on Noise Control Engineering (INTER-NOISE 75), Sendai, Japan, August 27-29 1975*. 85-92.

Kolsky, H. (1963). *Stress waves in solids*. 2nd ed. New York: Dover Publications, Inc.

Kurzweil, L.G. (1979). Ground-borne noise and vibration from underground rail systems. *Journal of Sound and Vibration*, **66(3)**, 363–370.

Lai. C.G., Callerio, A., Faccioli, E. Morelli, V. and Romani, P. (2005). Prediction of railway-induced ground vibrations in tunnels. *Journal of Vibration and Acoustics*, **127**, 503-514.

Lama, R.D. and Vutucuri, V.S. (1978). *Handbook on mechanical properties of rocks - testing techniques and results - Volume II*. 1st ed. Clausthal, Germany: Trans Tech Publications.

Leventhall, H.G. (1987). Low-frequency traffic noise and vibrations. In: *Transportation noise reference book*. Ed. by Nelson. P.M. London: Butterworths.

Melke, J. (1988). Noise and vibration from underground railway lines: proposal for a prediction procedure. *Journal of Sound and Vibration*, **120(2)**, 391–406.

Melke, J. and Kraemer, S. (1982). Diagnostic methods in the control of railway noise and vibration. *Journal of Sound and Vibration*, **87(2)**, 377–386.

Miklowitz, J. (1978). *The theory of elastic waves and waveguides*. Second printing. Amsterdam: North-Holland Publishing Company.

Nagy, A.B., Fiala, P., Márki, F., Augusztinovicz, F., Degrande, G., Jacobs, S. and Brassenx, D. (2006). Prediction of interior noise in buildings generated by underground rail traffic. *Journal of Sound and Vibration*, **293**, 680–690.

Nelson, J.T. (1996). Recent Developments in ground-borne noise and vibration control. *Journal of Sound and Vibration*, **193(1)**, 367–376.

NGI (2004). *New double tracked line Sandvika – Asker. Measures against structure borne noise*. 20041010-2. Oslo: Norges Geotekniske Institutt.

Nguyen, K-V. and Catmiri, B. (2007). Evaluation of seismic ground motion induced by topographic irregularity. *Soil Dynamics and Earthquake Engineering*, **27**, 183–188.

Möller, B., Larsson, R., Bengtsson, P-E. and Moritz, L. (2000). *Geodynamik i praktiken*. Linköping: Swedish Geotechnical Institute.

Paterson, M.S. (1978). *Experimental rock deformation - the brittle field*. Berlin Heiden-berg: Springer-Verlag.

Pretlove, A.J. and Rainer, J.H. (1995). Human response to vibrations. In: *Vibration problems in structures*. Ed. by Bachman, H. Basel: Birkäuser Verlag.

Ramana, Y.V. and Venkatanarayana, B. (1973). Laboratory Studies on Kolar Rocks. *International Journal of Rock Mechanics and Mining Science & Geomechanical Abstract*, 10, 465-480.

Remington. P.J., Kurtweil, L.G. and Tower. D.A. (1987). Low-frequency noise and vibrations from trains. In: *Transportation noise reference book*. Ed. Nelson. P.M. London: Butterworths.

Sahlin, S. and Sundqvist, H. (1995). *Banteknik*. Rapport 21, TRITA-BKN. KTH: Institutionen för byggkonstruktion. (In Swedish).

Schillemans, L. (2003). Impact of sound and vibration of the North–South high-speed railway connection through the city of Antwerp Belgium. *Journal of Sound and Vibration*, 267, 637–649.

SOSFS. (2005). Socialstyrelsens författningssamling – buller inomhus. SOSFS 2005:6 (M) (In Swedish).

Ungar, E.E. and Bender, E.K. (1975). Vibrations produced in building by passage of subway trains; parameter estimation for preliminary design. In: *Proceeding of the 4th International Conferences on Noise Control Engineering (INTER-NOISE 75), Sendai, Japan, August 27-29 1975*. 85-92.

SS-ISO 14837-1 (2005). M*echanical vibration – Ground-borne noise and vibration arising from rail systems - Part 1: General guidance.*

SS-ISO (1997). *Vibration and shock – Evaluation of human exposure to whole-body vibration – Part 1: General guidance.*

Unterberger, W., Poisel, R. and Honeger, C. (1997). Numerical prediction of surface vibrations caused by high-speed rail traffic in tunnels. In: *Proceedings of the 23rd general assembly of the International Tunnelling Association, Vienna, Austria, April 12-17 1997.*

WHO (1999). *Guidelines for community noise.* http://whqlibdoc.who.int/hq/1999/a68672.pdf (2007-11-19).

WSP (2005). *Gårdatunneln Göteborg – Studie av Stomljud från Tågtrafik.* Rapport 2005:9. (In Swedish).

Yang, Y.B. and Hsu, L.C. (2006). A review of researches on ground-borne vibrations due to moving trains via underground tunnels. *Advances in Structural Engineering*, **9(3)**, 377-392.

Youash, Y.Y. (1970). Dynamical physical properties of rock: part, II, Experimental Results. In: *Proceeding of the Second Congress of the International Society of Rock Mechanics, Belgrade, Yugoslavia, September 21-26 1970*. Vol. 1.