PD ISO/TS 14837-32:2015

BSI Standards Publication

Mechanical vibration — Ground-borne noise and vibration arising from rail systems

Part 32: Measurement of dynamic properties of the ground

...making excellence a habit.™

National foreword

This Published Document is the UK implementation of ISO/TS 14837-32:2015.

The UK participation in its preparation was entrusted by Technical Committee GME/21, Mechanical vibration, shock and condition monitoring, to Subcommittee GME/21/3, Mechanical vibration, shock and condition monitoring — Measurement and evaluation of mechanical vibration and shock in stationary structures.

A list of organizations represented on this committee can be obtained on request to its secretary.

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Published by BSI Standards Limited 2016

ISBN 978 0 580 83932 0
ICS 17.160; 45.080

Compliance with a British Standard cannot confer immunity from legal obligations.

This Published Document was published under the authority of the Standards Policy and Strategy Committee on 31 January 2016.

Amendments/corrigenda issued since publication

Date	Text affected

TECHNICAL SPECIFICATION

ISO/TS 14837-32

First edition
2015-12-15

Mechanical vibration — Ground-borne noise and vibration arising from rail systems —

Part 32: Measurement of dynamic properties of the ground

Vibrations mécaniques — Vibrations et bruits initiés au sol dus à des lignes ferroviaires —

Partie 32: Mesurage des propriétés dynamiques du sol

Reference number
ISO/TS 14837-32:2015(E)

Contents

Page

Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular the different approval criteria needed for the different types of ISO documents should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights. Details of any patent rights identified during the development of the document will be in the Introduction and/or on the ISO list of patent declarations received (see www.iso.org/patents).

Any trade name used in this document is information given for the convenience of users and does not constitute an endorsement.

For an explanation on the meaning of ISO specific terms and expressions related to conformity assessment, as well as information about ISO's adherence to the WTO principles in the Technical Barriers to Trade (TBT) see the following URL: Foreword - Supplementary information

The committee responsible for this document is ISO/TC 108, *Mechanical vibration, shock and condition monitoring*, Subcommittee SC 2, *Measurement and evaluation of mechanical vibration and shock as applied to machines, vehicles and structures*.

ISO 14837 consists of the following parts, under the general title *Mechanical vibration — Ground-borne noise and vibration arising from rail systems*:

— *Part 1: General guidance*

— *Part 32: Measurement of dynamic properties of the ground*

The following parts are under preparation:

— *Part 31: Measurement for the evaluation of complaints at residential buildings*

Introduction

In order to resolve received vibration and noise from rail systems where there is soil or rock in part of the transmission path between the source at the track and the receiver location in the building, it is necessary to know the noise and vibration transmission function of the ground. To know this necessitates knowledge of the properties of the materials in the ground and their stratification which influence the transmission. In general there is a need to measure or in other ways to estimate these properties. To this aim, this part of ISO 14837 defines methods for measurement and estimation of the relevant dynamic ground parameters.

After a brief survey about ground-borne noise and vibration in Clause 4, the key content of this part of ISO 14837 is outlined in two clauses. Clause 5 defines the relevant dynamic ground parameters, describes how they are interrelated and how they are related to basic physics of wave propagation. Clause 6 deals with methods to determine these parameters: 6.3 presents simple estimation methods based on empirical correlations with conventional geotechnical and engineering geological index parameters; 6.4 presents methods for indirect determination from geotechnical in-situ penetration test data, while 6.5 and 6.6 present more precise methods for direct measurement of the parameters in-situ and in the laboratory.

Mechanical vibration — Ground-borne noise and vibration arising from rail systems —

Part 32:
Measurement of dynamic properties of the ground

1 Scope

This part of ISO 14837 provides guidance and defines methods for the measurement of dynamic properties of the ground through which ground-borne noise and vibration is transmitted, from the operation of rail systems and into foundations of neighbouring buildings. The purpose is to determine the parameters of the ground system which are necessary to reliably predict the noise and vibration transmission, to design railroads and foundations to meet noise and vibration requirements, to design countermeasures and to validate design methods.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 14837-1:2005, *Mechanical vibration — Ground-borne noise and vibration arising from rail systems — Part 1: General guidance*

3 Symbols

The following symbols are used in this part of ISO 14837.

NOTE Abbreviations are summarized in Annex A.

B dimensionless constant in equation for G_{max}

D loss-related distance attenuation factor

d distance travelled by wave

E_{max} Young's modulus, low-strain dynamic value

e void ratio, $e = \varphi/(1 - \varphi)$

f frequency

G^* complex shear modulus

G_{max} shear modulus, low-strain dynamic value

G_{sec} secant shear modulus, dynamic value

I_p plasticity index

K_{max} bulk modulus, low-strain dynamic value

k^* complex wave number

M^*	complex constrained modulus
M_{max}	constrained modulus, low-strain dynamic value
N_{60}	corrected blow count from standard penetration test (SPT)
n	stress exponent in equation for G_{max}
P	vibration power flux
p_a	reference stress (pressure), $p_a = 100$ kPa
Q	material quality factor
Q_c	rock quality factor
q_t	tip resistance from cone penetration test (CPT)
R	radial distance, slanted or along surface
R_0	reference radial distance, slanted or along surface
S_{PP}	power spectrum of wave power flux
S_r	degree of saturation
S_{vv}	power spectrum of vibration particle velocity
s_u	undrained shear strength
t	time
V	wave speed independent of wave type
$\overline{V}$	average wave speed
V_p	compression wave speed
V_s	shear wave speed
V_{water}	sound (wave) speed in water
v	particle velocity of vibration
v_0	particle velocity of vibration at reference distance
v_{RMS}	root-mean-square value of vibration particle velocity
W	potential energy in a hysteresis loop
Z_p	specific impedance for plane compression waves
Z_s	specific impedance for plane shear waves
z	depth coordinate, depth below ground surface
α	distance attenuation exponent
γ_{cy}	cyclic (dynamic) shear strain
Δt	time interval, time difference
ΔW	energy loss in one hysteresis loop

ε_{cy}	cyclic (dynamic) normal strain
ζ	degree of critical damping (damping ratio) of SDOF
η	material loss factor
η_p	material loss factor for compression waves
η_s	material loss factor for shear waves
η_{max}	material loss factor at very small strains, in linear range
λ	wavelength
λ_{max}	1st Lamé constant, low-strain dynamic value
μ_{max}	2nd Lamé constant, low-strain dynamic value
ν_0	Poisson's ratio, low-strain value
ξ	damping ratio, $\xi = \eta/2$
ρ	bulk mass density
$\rho_{mineral}$	mass density of mineral grains
ρ_{water}	mass density of water
σ_{cy}	cyclic (dynamic) normal stress
σ'_{mean}	mean effective stress
σ'_{v}	effective vertical normal stress
τ_{cy}	cyclic (dynamic) shear stress
φ	porosity

4 Transmission of ground-borne noise and vibration

4.1 General

Ground-borne noise and vibration from rail systems are transmitted through the subsurface as mechanical waves. The wave propagation is influenced both by geometrical parameters like shape, extent and stratification of the various bodies of the ground, as well as by the dynamic properties of the individual ground materials.

To determine the influence of the geometrical parameters, it is necessary to understand wave types, wave speeds and impedances which are again dependent on properties of the soils and rocks involved. The distance attenuation of ground-borne noise and vibration is largely controlled by the geometrical effects in addition to the effect of loss mechanisms in the various ground materials.

For ground-borne noise and vibration from rail systems, the typically appearing dynamic strains in the ground are predominantly low and mostly within the range where the materials behave linearly. Elastic wave propagation can therefore be assumed. However, even at these small strains, ground materials do expose some internal energy loss, materializing as a contribution to the distance attenuation. Slightly viscoelastic or hysteretic rather than purely elastic behaviour do therefore better describe ground materials for this purpose.

In an unbounded, homogeneous elastic medium only two fundamental wave types can exist; dilatational waves (p-waves) and shear waves (s-waves). Dilatational waves have their particle motion back and forth along the direction of wave propagation while in shear waves particles move perpendicular to the direction of propagation. Shear waves can therefore be polarized in various planes while p-waves are un-polarized. A real ground is far from unbounded and homogeneous. It has free surfaces and interfaces between various layers and bodies of ground materials with different properties. The dilatational and shear waves in these layers and bodies of the ground are transmitted, reflected and refracted and in other ways interact to form different kinds of inferred waves like surface waves and guided waves along the surfaces and interfaces in the ground. These waves can often carry most of the vibration energy and thus control the distance attenuation. These types of waves are usually dispersive, meaning that vibration propagation properties can vary largely with frequency and wave length.

Since this whole complex of wave types that make up the vibration transmission path through a real ground all originate from the two fundamental waves, the information needed to resolve the transmission wave field is limited to the ground material properties that govern these fundamental waves, in addition to how these properties are distributed throughout the ground. From these properties and the geometry of their distribution in layers and bodies of the ground throughout the transmission path, the whole wave field can be solved, at least theoretically. In practice, however, major simplifications are usually necessary to be able to come up with reasonable solutions for real cases.

Complementary information on these issues are found in Reference [105].

4.2 Ground-borne noise versus vibration — Effect of frequency

What is termed ground-borne vibration and ground-borne noise are both transmitted through the ground and into buildings as mechanical vibration waves. The same physical wave mechanisms and fundamental waves do therefore apply to both noise and vibration. The only distinction is the frequency and thus the wave length. According to ISO 14837-1, the relevant frequency range of ground-borne vibration from rail systems is defined to be the range of whole-body vibration perception, which goes from 1 Hz to 80 Hz (see ISO 8041). The relevant frequency range of ground-borne noise from rail systems is in the audible range and is in ISO 14837-1 considered from about 16 Hz to 250 Hz. Ground-borne noise is reradiated as sound from vibrating building surfaces, while ground-borne vibration is transmitted to the whole body primarily from vibrating floors. In addition to affecting people, ground-borne vibration may also have an effect on sensitive installations and even on building structures.

Even though the basic wave propagation physics is the same, geometrical effects, secondary waves and loss mechanisms usually dominate the ground transmission of mechanical vibration, and introduce frequency-dependent transmission properties. The transmission of ground-borne noise at a given site can therefore be drastically different from the transmission of low-frequency vibration.

5 Parameters for wave propagation in the ground

5.1 General

Clause 5 gives an overview of the most important material parameters for transmission of ground-borne noise and vibration and how they are theoretically composed and interrelated. Clause 6 presents methods to quantify these parameters through empirical estimation and measurement.

5.2 Fundamental wave propagation parameters

In an elastic, isotropic, homogeneous solid, two fundamental plane body wave types can exist: the dilatational wave (p-wave, compression wave) and the shear wave (s-wave). The propagation speed V of these waves is related to the stiffness moduli and bulk (effective) mass density of the soil and rock through which they propagate, according to the following:

a) propagation speed, V_p, of dilatational wave

$$V_p = \sqrt{\frac{M_{max}}{\rho}} \tag{1}$$

b) propagation speed, V_s, of shear wave:

$$V_s = \sqrt{\frac{G_{max}}{\rho}} \tag{2}$$

where

M_{max} is the elastic constrained modulus;

G_{max} is the elastic shear modulus of the medium;

ρ is the bulk (effective) mass density.

By using Pa as unit for the moduli and kg/m^3 for the mass density, the wave speeds from these formulae appear with m/s as the unit.

The subscript max comes from the terminology of soil and rock dynamics. It denotes the maximum, stable plateau value reached for the respective moduli when the dynamic strains are sufficiently low to make the ground materials behave linearly elastic. At higher dynamic strains, non-linearity leads to a reduced secant modulus compared to this maximum value. It is vital to distinguish these dynamic moduli from moduli provided for use in conventional static or quasi-static soil and rock mechanics. Those moduli are determined for much higher stresses (strains) and for more long-term (permanent) loads where non-linearity and creep play an important role. This usually leads to drastically lower moduli than their linear dynamic counterparts. Applying static moduli in rock and soil dynamic calculations can lead to severely incorrect results.

The deformation properties of an isotropic, elastic medium are in general uniquely defined by two independent elastic parameters. These may be M_{max} and G_{max}. By instead introducing Poisson's ratio, ν_0, as the second parameter, the constrained modulus, M_{max}, can be expressed from the shear modulus, G_{max}, through Formula (3):

$$M_{max} = \frac{2(1-\nu_0)}{1-2\nu_0} G_{max} \tag{3}$$

The Poisson's ratio used here with subscript 0 is the one that applies to low dynamic strains in the linear elastic regime and can be largely different from the quasistatic counterpart used in soil and rock mechanics.

The dilatational wave speed, V_p, can further be expressed from the shear wave speed, V_s, and Poisson's ratio, ν_0, as Formula (4):

$$V_p = \sqrt{\frac{2(1-\nu_0)}{1-2\nu_0}} V_s \tag{4}$$

It appears from Formula (4) that V_p is always larger than V_s, i.e. $V_p > V_s$, and that V_p/V_s increases drastically if ν_0 approaches 0,5.

Poisson's ratio ν_0 can be determined from the speeds of the two fundamental body waves, V_p and V_s, by Formula (5):

$$\nu_0 = \frac{V_p^2 - 2V_s^2}{2\left(V_p^2 - V_s^2\right)} \tag{5}$$

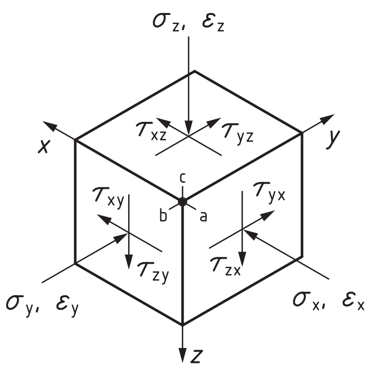
If the Young's modulus E_{max} is instead introduced as the second elastic parameter it is related to the shear modulus G_{max} and Poisson's ratio ν_0 through Formula (6):

$$E_{max} = 2\left(1 + \nu_0\right)G_{max} \tag{6}$$

Alternatively, the bulk modulus K_{max} can be introduced as one of the two elastic parameters. Its interrelation to the others reads:

$$K_{max} = M_{max} - \frac{4}{3}G_{max} \tag{7}$$

Figure 1 specifies the stress and deformation quantities as well as the coordinate axes in the ground. Compression is positive, i.e. $\sigma > 0$. The first subscript of the shear quantity τ is the direction and the second is the plane.



Key

a	x-plane	$\Delta\tau_{xy} = \Delta\tau_{yx}$
b	y-plane	$\Delta\tau_{yz} = \Delta\tau_{zy}$
c	z-plane	$\Delta\tau_{zx} = \Delta\tau_{xz}$

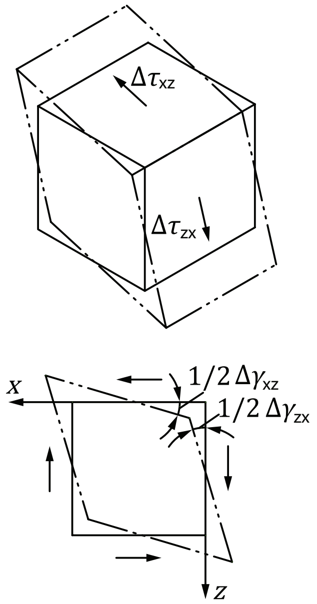
Figure 1 — Stress quantities and coordinate axes in the ground

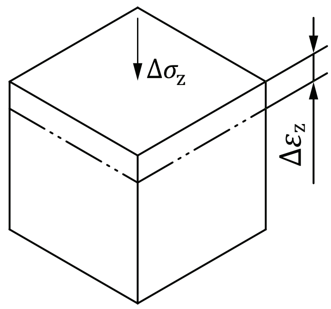
Figure 2 illustrates the stress and deformation modes of change for which each of these moduli does apply. An alternative set of elastic parameters are the two Lamé constants.

NOTE 1 The two Lamé constants are an alternative set of elastic parameters. They are uniquely related to those already defined. When used for soil and rock materials, it is also convenient to denote the low-strain plateau values of these parameters by the subscript max, even if that is not so commonly used. The Lamé parameters relate to the other parameters through the formulae:

$\lambda_{max} = M_{max} - 2G_{max}$

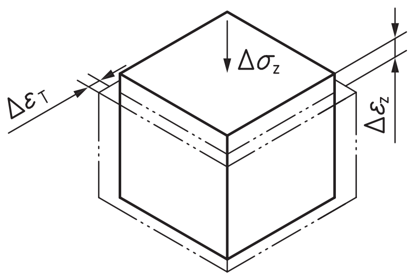
$\mu_{max} = G_{max}$

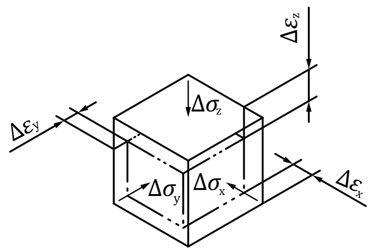




a) Shear modulus $G = \Delta\tau/\Delta\gamma$

b) Constrained modulus $M = \Delta\sigma_z/\Delta\varepsilon_z$





c) Young's modulus $E = \Delta\sigma_z/\Delta\varepsilon_z$

d) Bulk modulus $K = \Delta p/\Delta\varepsilon_{vol}$

a) $\Delta\tau_{xz} = \Delta\tau_{zx} = \Delta\tau$, $\Delta\gamma_{xz} = \Delta\gamma_{zx} = \Delta\gamma$.

b) Sides restricted $\Delta\varepsilon_x = \Delta\varepsilon_y = 0$, no transverse strain.

c) $\Delta\varepsilon_x = \Delta\varepsilon_y = \Delta\varepsilon_T$, sides free $\Delta\sigma_x = \Delta\sigma_y = 0$, Poisson's ratio $\nu = \Delta\varepsilon_T/\Delta\varepsilon_z$.

d) $\Delta\sigma_x = \Delta\sigma_y = \Delta\sigma_z = \Delta p$, $\Delta\varepsilon_x + \Delta\varepsilon_y + \Delta\varepsilon_z = \Delta\varepsilon_{vol}$.

NOTE The illustrations are made for the *z*-direction being the active direction. The definitions equally apply as well for the *x*-direction or the *y*-direction being the active direction.

Figure 2 — Elastic constants and associated deformation modes for isotropic ground

Specific impedance of ground materials and impedance contrasts between materials are important parameters for evaluating the amount of wave energy transferred over interfaces from one ground material body into another and for the formation of interface waves along the boundaries between ground material bodies. It is, however, essential to be aware that for thin layers compared to the vibration wavelength, it is rather the thickness of the layers, and not the impedance contrasts that control the reflection and transmission of vibrations. This is particularly important when assessing the effect of typical vibration isolation screens.

The specific impedances, Z, for plane waves in an elastic isotropic ground material body are defined as follows:

a) specific impedance, Z_p, for plane dilatational waves

$$Z_p = \rho V_p = \sqrt{\rho M_{max}} \tag{8}$$

b) specific impedance, Z_s, for plane shear waves

$$Z_s = \rho V_s = \sqrt{\rho G_{max}} \tag{9}$$

The inverse of the specific impedance is termed specific admittance and may be used alternatively.

The specific impedance has the unit Pa/(m/s) and relates dynamic stress (cyclic stress) in a propagating wave to the corresponding particle velocity. A plane shear wave with particle velocity v which propagates in one direction through a ground material with specific shear wave impedance, Z_s, imposes dynamic shear stress, τ_{cy}, in the plane of wave polarization equal to Formula (10):

$$\tau_{cy} = Z_s v \tag{10}$$

The corresponding shear strain, γ_{cy}, is:

$$\gamma_{cy} = \frac{v}{V_s} \tag{11}$$

Corresponding relations apply for normal stress, σ_{cy}, and normal strain, ε_{cy}, in the direction of propagation of a dilatational wave with particle velocity v:

$$\sigma_{cy} = Z_p v \tag{12}$$

$$\varepsilon_{cy} = \frac{v}{V_p} \tag{13}$$

Formulae (10) to (13) do only apply to one single fundamental wave component travelling in one direction. Where more wave components interact like in standing waves, surface and interface waves, superposition need to be considered and the above relations cannot be applied immediately.

The average mechanical power density flux, P, in W/m^2, transmitted in the direction of a propagating fundamental plane wave component at a single frequency is:

$$P = \frac{1}{2} Z \hat{v}^2 \tag{14}$$

where

Z is the specific impedance of the ground for the wave type in question;

$\hat{v}$ is the particle velocity amplitude of that single frequency component of the wave.

Correspondingly, for broad-band vibration propagation, the spectral density, S_{PP}, of the power flux is:

$$S_{\mathrm{PP}}(f) = \frac{1}{2} Z\, S_{\mathrm{vv}}(f) \tag{15}$$

where $S_{vv}(f)$ is the power spectral density function of the particle velocity of the ground vibration propagated by the wave. By applying $(m/s)^2$/Hz as unit for $S_{vv}(f)$, the resulting unit for $S_{PP}(f)$ is (W/m^2)/Hz.

For a broad-band vibration with an r.m.s. value of particle velocity, v_{RMS}, the total power flux P_{tot} is:

$$P_{\mathrm{tot}} = \frac{1}{2} Z v_{\mathrm{RMS}}^2 \tag{16}$$

Vibration events from rail systems are transient and time varying. When it comes to representative values of particle velocities, strains, stresses and power, a form of running r.m.s. time domain amplitudes or running spectral values can be the most relevant as pointed out in ISO 14837-1:2005, 7.4. Further, from experience, a 1 s running time window analysis gives the most representative values for evaluation of the dynamic performance of soil and rock materials.

For resolving low frequencies around 1 Hz, a longer integration time is needed, typically 3 s to 5 s. However, this should be implemented with care, ensuring that the low-frequency content of the signal is sufficiently stationary over this period of time.

Strictly, the above formulae are valid for plane waves only. However, they can be used as good approximations for the real waves that appear in the handling of ground-borne noise and vibration related to rail systems. Only in particular situations like close to the vibration source, i.e. in the nearfield, spherical wave theory might need to be considered. Nearfield effects are further discussed in 5.5.

NOTE 2 For theory on spherical waves, see Reference [90].

5.3 Material loss and non-linearity

For ground-borne noise and vibration from rail systems, the dynamic strains in the ground are mostly within the range where the materials have a nearly linear behaviour. However, even at small strains, ground materials do expose some energy loss which materialize as a small amount of attenuation. Only close to the source of dynamic loads in the track structure, in close vicinity to the track, at sharp edges and discontinuities somewhat higher dynamic strain can appear. For critical train speed and train speeds exceeding the Rayleigh wave speed in the ground (trans-Rayleigh), excessive track and ground vibration can appear and the strains can be so high that non-linearity needs to be accounted for to simulate, understand and mitigate the condition.[102] For ground-borne noise and vibration, materials can conveniently and with sufficient accuracy be modelled as viscoelastic (Kelvin-Voigt model), with a material loss factor, η, defined as:

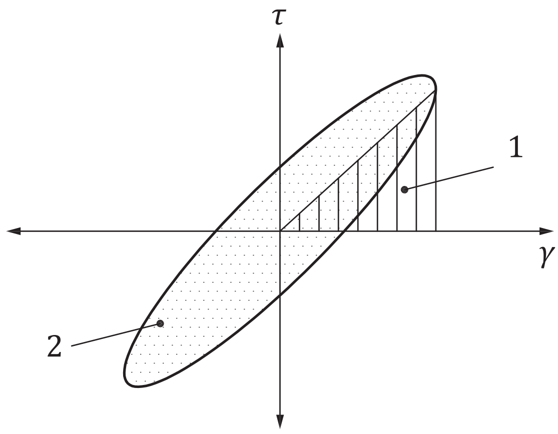
$$\eta = \frac{1}{2\pi} \cdot \frac{\Delta W}{W} \tag{17}$$

where

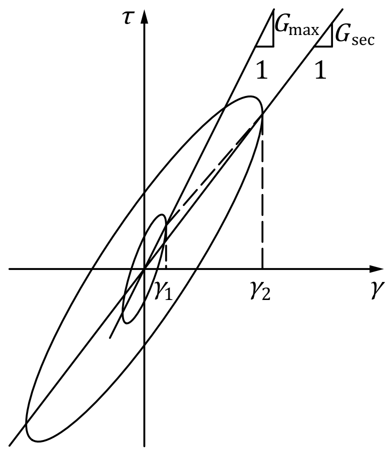
W is the potential energy in a hysteresis loop (load cycle);

ΔW is the energy loss in one load cycle.

The material loss factor quantifies the energy loss in each load cycle as a vibration wave passes by, as shown in Figure 3.



a) Definition of hysteretic damping ratio $\xi = \eta/2 = \frac{1}{4\pi} \cdot \frac{\Delta W}{W}$



b) Definition of G_{max} **and** G_{sec}

Key

1 peak potential energy during cycle, W

2 energy dissipated in one cycle, ΔW

γ_1 strain amplitude in linear range, i.e. very small strain

γ_2 strain amplitude in non-linear elastic range, or above (see Figures 8 and 10 for definition of strain ranges)

NOTE The elliptic shaped hysteresis applies to viscous damping while it has sharp edges for damping of hysteretic or frictional nature.

Figure 3 — Definition of hysteretic damping quantities for ground materials

The material loss factor can take different values for shear and dilatational waves, and is then termed η_s and η_p respectively. Other disciplines like seismology and geology traditionally quantify the loss property of ground materials by the quality factor Q. The quality factor Q and the loss factor η are related through:

$$Q = \frac{1}{\eta} \tag{18}$$

In literature on soil dynamics and geotechnical earthquake engineering,[20] the hysteretic loss in ground materials is often quantified as a term called damping ratio or damping factor.

NOTE 1 The damping ratio or damping factor used in literature on soil dynamics and geotechnical earthquake engineering is commonly termed ξ and is defined from the energy loss in the hysteretic loops in the same way as the loss factor η. The damping ratio is, however, defined as $\xi = \frac{1}{4\pi} \cdot \frac{\Delta W}{W}$, which makes $\xi = \eta / 2$. The damping ratio ξ in this context is a hysteretic material property and should not be mixed up with the fraction of critical damping or damping ratio of a mechanical system like a SDOF system, ζ. The fraction of critical damping is defined as $\zeta = C / C_{cr} = C / \left(2\sqrt{km}\right)$ where C, C_{cr}, k and m are the damping constant, critical damping constant, spring constant and mass of the SDOF system respectively. In an SDOF system, the coupled spring-damper will exert hysteretic loops with increasing $\frac{\Delta W}{W}$ and thus increasing ξ for increased frequency. However, at its natural frequency, and only there, the hysteretic damping ratio, ξ, determined from $\frac{\Delta W}{W}$ of this hysteresis loop is the same as the fraction of critical damping for the system, ζ, determined from its C, k and m values.

The loss in ground materials is of frictional and hysteretic nature (slightly non-linear) rather than viscous and is thus nearly insensitive to frequency. To model hysteretic loss and still maintain the convenience of a linear viscoelastic formulation, a viscosity that is inversely proportional to the frequency is introduced. This eliminates frequency dependency inherent in viscoelasticity which is not relevant for hysteretic material behaviour. This loss formulation is often termed linear hysteretic.[20] Implemented in the Kelvin-Voigt formulation, this leads to complex stiffness moduli:

a) complex stiffness modulus M^* for dilatational waves

$$M^* = M(1 + \mathrm{i}\eta_p) \tag{19}$$

b) complex stiffness modulus G^* for shear waves

$$G^* = G(1 + \mathrm{i}\eta_s) \tag{20}$$

where

$\mathrm{i} = \sqrt{-1}$

The complex moduli do also lead to complex wave speeds and wave numbers. All the wave propagation formulae presented above do hold true also when replacing the conventional real moduli with complex moduli.

NOTE 2 The complex wave speeds and corresponding complex wave numbers can be expressed as $V^* = V_1 + \mathrm{i}V_2$ and $k^* = k_1 + \mathrm{i}k_2$ respectively. Here k_2 is the term representing the loss attenuation (damping) with distance of wave propagation and is related to the loss factor η through $k_2 = -\frac{2\pi f}{V}\sqrt{\frac{\sqrt{1+\eta^2}-1}{2\left(1+\eta^2\right)}}$ where V is the propagation speed of the wave type in question. More elaboration on complex wave speeds and wave numbers is found in Reference [20] and [91].

The contribution from the loss factor of the ground material to the attenuation of a vibration wave as it propagates from distance R_0 to R is often expressed by Formula (21):

$$\mathrm{e}^{-\alpha\left(R-R_0\right)} \tag{21}$$

α is usually considered an empirical factor matched to the site and the situation. However, based on the complex moduli and wave numbers, the α factor can be directly related to the material loss factor, through Formula (22):

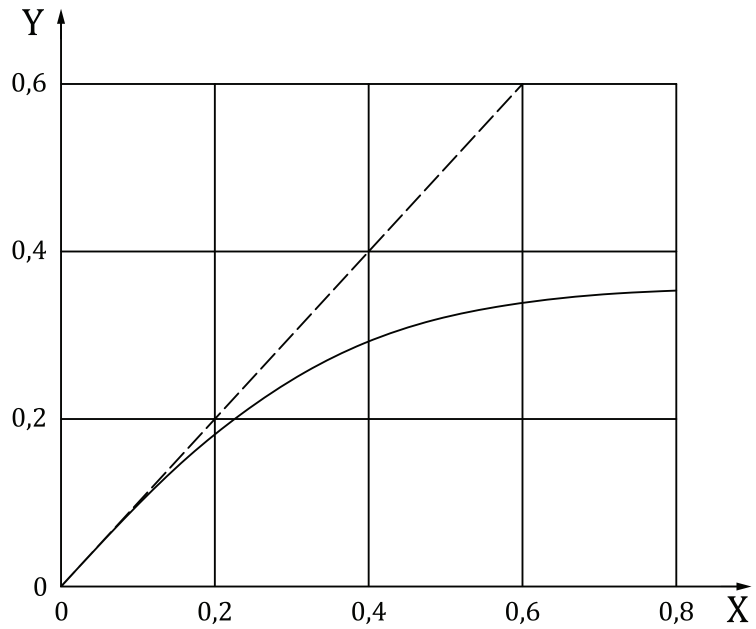
$$\mathrm{e}^{-\alpha\left(R-R_0\right)} = \mathrm{e}^{-2\pi f D\left(R-R_0\right)/V} = \mathrm{e}^{-2\pi D\left(R-R_0\right)/\lambda} \tag{22}$$

where

- V is the propagation speed of the wave type in question;
- f is the frequency;
- λ is the corresponding wavelength;
- D is the loss-related distance attenuation factor.

For low loss factors, D can be approximated by $D = \eta/2$. This approximation holds true as long as η is lower than about 0,3 as can be seen from Figure 4 where the exact relation is plotted. The difference $R - R_0$ is the actual distance travelled for the wave type in question. As can be seen from Formula (22), there is a constant fraction of loss attenuation for each wave length the vibration wave has travelled.

NOTE 3 The exact relation between D and η reads: $D = \sqrt{\frac{\sqrt{1+\eta^2}-1}{2\left(1+\eta^2\right)}}$. This formula is valid for any loss factor and is plotted in Figure 4. From this formula, D can never take a higher value than about 0,35.



Key

X hysteretic damping ratio, *ξ*

Y distance attenuation, *D*

NOTE Numbers need to be multiplied by 100 to give damping values in %.

Figure 4 — Loss-controlled distance attenuation, *D*, versus hysteretic damping ratio, *ξ*

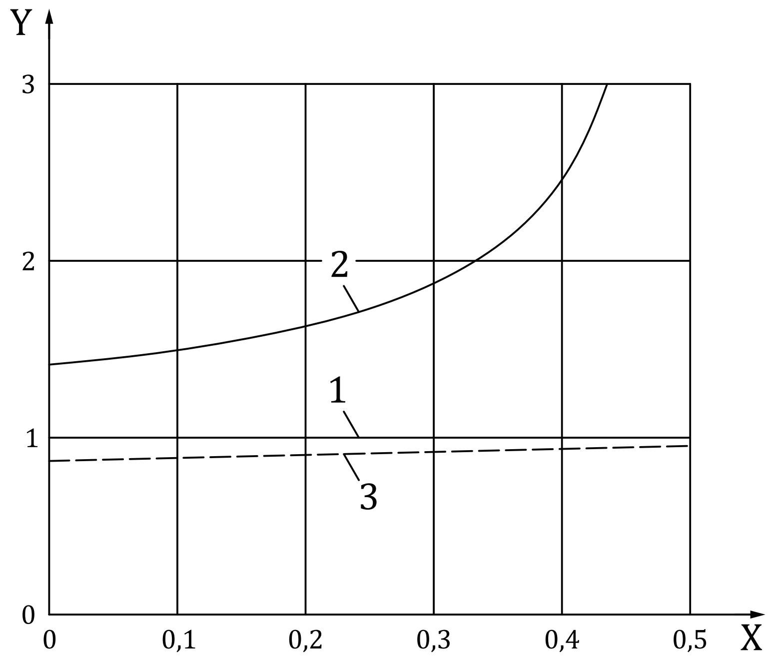
5.4 Geometry effects, stratification and lateral variability of the ground

Geometrical effects usually surpass loss effects in distance attenuation of ground-borne noise and vibration, particularly at low frequency. The fundamental geometric effect, often termed radiation, distance or geometric attenuation, is due to the fact that the wave energy is distributed over an increasingly larger surface as waves spread out as they propagate away from the source. This leads to a decrease in mechanical power density and thus a decrease in vibration amplitude with increasing distance. Basically, geometric attenuation leads to the amplitude of the fundamental body waves to decrease proportionally to the distance when radiated from a point source within an isotropic, homogeneous medium. Attenuation due to material loss mechanisms adds to this.

NOTE 1 Sometimes the term damping or radiation damping is used, although it is preferable to use the term geometric attenuation.

For a homogenous elastic half-space, one surface wave can exist in addition to the two fundamental body waves. The surface wave, often termed Rayleigh wave, has a speed V_R, slightly lower than that of shear waves in the same material. The ratio V_R/V_s varies between 0,87 and 0,96 depending on the Poisson's ratio. Figure 5 plots V_R/V_s and also V_p/V_s versus the Poisson's ratio.

Rayleigh waves propagate along the free surface with retrograde elliptical particle motion which dies out with increasing depth. Most wave motion is confined in a layer about one wave length deep. This means that low-frequency waves penetrate deeper into a half-space than higher-frequency waves. The Rayleigh wave energy from a point source thus spreads out on a cylindrical surface, rather than spherical surface. This geometric energy spreading therefore leads to an amplitude attenuation for these waves which is proportional to the square root of the distance.



Key

1 shear wave (s-wave)

2 p-wave

3 Rayleigh wave

X Poisson's ratio

Y wave propagation speed normalized with respect to shear wave speed

Figure 5 — Speed of Rayleigh and p-waves normalized to the shear wave speed, plotted versus Poisson's ratio

Surface waves are in many cases of major importance for ground-borne vibration and noise from rail systems. Firstly, most of the energy from a surface vibratory source is converted into surface waves. Secondly, the surface waves attenuate less with distance than body waves and are therefore also more dominant at larger distances.

For a homogeneous half-space, the total distance attenuation covering both geometrical and loss attenuation can be expressed as in Formula (23):

$$\frac{v}{v_0} = \left(\frac{R_0}{R}\right)^n \mathrm{e}^{-\alpha\left(R-R_0\right)} \tag{23}$$

NOTE 2 How the attenuation factor α relates to the material loss factor, the frequency and the wave type and wave speed is described in 5.3.

In Formula (23), v_0 and v are the vibration particle velocities at distances R_0 and R from the source, respectively. For a homogeneous half-space, the exponent n is 1 for p- and s-body waves within the medium, 0,5 for surface Rayleigh waves and 2 for the surface response to the body waves. The first term accounts for the geometrical attenuation, and is the same for all frequencies. The second term accounts for loss attenuation as explained above. This attenuation is higher for high frequency than for low. From its nature, Formula (23) is only relevant within a region where the vibration transmission is dominated by one single wave type only. If there is a dominant propagation of body waves from a tunnel, the $R - R_0$ term is the slanted distance. On the other hand, if the propagation is dominated by surface waves, $R - R_0$ is the distance along the surface.

Formula (23) does, however, represent a severe oversimplification when coming to real cases. Stratification and inhomogeneity in real ground in addition to nearfield effects can severely contradict the predictions by this formula.

Real ground is far more complicated than a homogeneous half-space. It is geometrically stratified, with layers and bodies of different ground materials and material properties, is inhomogeneous laterally, as well as vertically, and is often anisotropic. Under such conditions, surface waves become dispersive and exhibit several wave modes. A dispersive wave has a frequency- and wavelength-dependent propagation speed. Many other wave types and modes can appear along layer interfaces and guided within layers. To what extent the various wave types can exist, how much wave energy they attract and how they are attenuated depends on the frequency and the wavelength. Wave energy can be scattered by inclined layers and interfaces, and by joints when propagating through rock. Vibration at various frequencies and particularly ground-borne noise versus vibration can therefore propagate and attenuate very differently in such a real ground.

If stratification and fundamental dynamic properties of the involved soil and rock materials is known, wave propagation in such a complicated medium can to some extent be modelled by computational tools. However, drastic simplifications are usually needed to make the problem solvable in a manageable way. In this respect, it is important to be aware of the fact that layer boundaries are in reality fuzzy and can be undulating and inclined. Such features can disrupt resonances and destroy wave guide abilities for wave propagation through the ground. This can appear as excessive additional damping in the real world, but can be lost in an idealized computational model, which might therefore make the model tend to over predict the vibration transfer if these effects are not properly accounted for.

Rail-bound trains and trams are for most cases much longer than the distance to the response points of concern. The individual sources within the trains are, however, mostly not coherent. A train does therefore not act as a coherent line source nor as a point source, but rather like a long line of non-coherent sources. For discrete points along the railway line, the spacing which leads to coherent vibration is longer for low than for high frequencies.

5.5 Nearfield effects

Close to the vibration source and close to major wave reflectors there exists a nearfield where the wave pattern is more complicated than further away. In the nearfield, shear and dilatational waves are not separable and behave as one integrated wave phenomenon. The nearfield gradually degrades into a farfield over a distance of about three wavelengths from the source. The extent of the nearfield can therefore be large when the frequency is low. The distance attenuation is not monotonic and is more irregular in the nearfield than it is in the farfield, even for homogeneous ground conditions.

5.6 Anisotropy

Soils and rocks do usually have anisotropic wave propagation properties, either global anisotropy or material anisotropy. Global anisotropy is due to stratification, faulting etc., while material anisotropy may be due to anisotropic orientation of mineral particles, anisotropic stress conditions etc.

Global anisotropy may significantly influence transmission of ground-borne noise and vibration from rail systems. Scattering and diffraction leads to excess attenuation and layering opens for interface waves with frequency dependent and variable attenuation. Reliable prediction and design to handle ground-borne noise and vibration therefore need to account for these effects. This topic is further discussed in 5.4.

Material anisotropy will usually be weaker than global anisotropy and has less influence on wave propagation. This anisotropy is harder to quantify either by in-situ tests or in laboratory samples, and is more complicated to model. Material anisotropy is therefore often disregarded when dealing with ground-borne noise and vibration from rail systems.

When making direct, in-situ full-scale measurement of noise and vibration propagation properties of the ground at a site, the effect of various anisotropies of the ground at the site will inherently be determined. However, if generalizing this type of measurement results to be applied to other sites, differences in anisotropy among the sites may be a significant source of error.

5.7 Ground water effects — Ground materials as a two-phase medium

Water saturation has a large effect on wave propagation properties of soils, particularly if they are soft. Also in fractured rock, saturation has an effect but less pronounced. Strictly speaking, a wet soil composed of a matrix of mineral particles with the voids between them, fully or partly saturated with water obeys the theory of a two- or three-phase medium (e.g. Biot theory for poroelasticity [40]). However, for ground-borne noise and vibration from rail systems, the frequencies of importance are low and the corresponding wave length are long compared to the particle structure of the ground. It is therefore fully satisfactory to disregard the two-phase and particulate nature and smear the properties out and make them represented by an effective continuous medium, defined by corresponding continuum parameters as described in the preceding subclauses.[108]

It is, however, important to be aware that due to the low compressibility of water compared to the grain matrix of soft soils, the dilatational wave speed, V_p, of fully saturated loose soils usually turns out to be about 1 500 m/s, which is at least one order of magnitude more than the speed of shear waves, V_s, in such soils. This makes the apparent effective medium Poisson's ratio ν_0 close to 0,5 which can pose a problem for some numerical calculation tools. It is further important to be aware that this situation requires the soil to be completely saturated. Even a minute amount of air or gas in the pore water drastically reduces the effective medium V_p to a value close to $V_p = 2\ V_s$, see also Formula (28) and 6.3. More details on the wave speed in saturated soils are found under fluid substitution theory. For the same reasons, estimation of s-wave speed from a measured p-wave speed will be inappropriate. In cases where waves propagate from saturated porous medium to unsaturated media, the generation of wave-related elasticity to poroelasticity, e.g. the Biot slow p-wave, can appear and lead to excess attenuation due to loss mechanisms not otherwise accounted for.

NOTE Details on modelling the effect of pore fluid on the wave speeds in ground materials is found under the Biot-Gassmann fluid substitution theory, further described in Reference [39]. See also References [40], [41] and [42].

Varying ground water table and river level with season can in this way change vibration propagation characteristics. However, a seasonally varying water table does not ensure that all mineral surfaces are re-wetted and the soil is re-saturated to a sufficient degree to behave as fully saturated with respect to p-wave speed. The significance of taking saturation into account is to get the p-wave speed and the bulk density correct. The p-waves are mainly an issue for underground rail systems. For surface rail systems on the contrary, s-waves and surface waves dominate the noise and vibration ground propagation, and these waves are rather insensitive to water saturation, but largely effected by the depth of the ground water table.[106]

6 Methods for parameter estimation and measurement

6.1 Stratification and classification of the ground: Boring logs and seismic investigations

Before making any assessment of ground-borne noise and vibration for a rail system project, basic knowledge about the ground conditions and stratification at the site is crucial. Information about soil types, layering, lateral extent, depth to bed rock or hard base, rock types and properties, and depth to the ground water table is essential.

For most cases, this basic information is already made available from standard site investigations. These may already have been performed for the purpose of geotechnical or rock mechanical design at the site, independent of the ground-borne noise and vibration issue. It is, however, important that the noise and vibration expert gets full access to the geotechnical and geological information and understands how to utilize it. Preferably, the noise and vibration expert should be involved also in the design of the standard site investigation to ensure its extent is sufficient, as the sphere of noise and vibration influence often extends beyond the sphere of importance for geotechnical or rock mechanical design. Supplementary borings and investigations particularly addressing noise and vibration issues can more efficiently be included at the early planning stage. Methods for general geotechnical and engineering geological site investigation are specialized and there are a variety of local adaptations for various countries and regions. That topic is beyond the scope of this part of ISO 14837. An overview of the most common investigation methods is found in textbooks like in References [14] and [19]. An overview of commonly used geotechnical and rock mechanical index parameters is found in Reference [16].

Besides giving general information about type and extent of soils and rock at the site, data from the conventional geotechnical and rock mechanical investigations reveal index parameters that can be useful input to estimate also the dynamic properties of the ground and thus be used for the noise and vibration calculations.

Typical standard site investigation methods are general sounding (with many local variations), cone penetration testing (CPT), standard penetration testing (SPT), sampling and laboratory testing, in addition to conventional refraction seismic methods. Important index parameters that are useful for dynamic analyses are porosity (or void ratio), degree of water saturation, undrained shear strength, plasticity index, over-consolidation ratio and rock mass quality designation. Methods to estimate dynamic ground parameters from index parameters are presented in 6.3. 6.4 present methods to indirectly determine dynamic parameters from in-situ penetration testing (CPT and SPT). More reliable data are obtained by using dedicated in-situ methods for direct measurement of dynamic properties of the ground; 6.5 describe these methods. Soil and rock cores retrieved by geotechnical or rock mechanical drilling operations can be tested in the laboratory to directly obtain data on dynamic properties in addition to the primary geotechnical and rock mechanical data. Dedicated laboratory methods for measuring dynamic properties in retrieved samples are described in 6.6.

6.2 Soils versus rock

For determining dynamic parameters for the ground, it is important to be aware that soils and rocks behave distinctly different under most geological conditions. Different methods are therefore conveniently used to assess their dynamic properties. Soils and rock are therefore treated separately in the following subclauses.

One should, however, be aware that under some geologic and climatic conditions the distinction between soil and rock is not always clear. This is often the case in tropical areas with residual soils. These soils have a more gradual transition from loose soils to firmer rock. For residual soils, the clear distinction between mineral grains and void space might neither be valid, as the grains themselves can be fractured, absorb water and completely change nature when wetted.

6.3 Empirical estimation methods based on index parameters

6.3.1 General

There exist a large amount of empirical and semi empirical methods to estimate dynamic properties of ground materials based on general geotechnical and rock mechanical index parameters and other data from conventional site investigations and laboratory testing. This subclause presents a set of such prediction formulae. The literature does, however, contain alternative equations that can give better estimates for particular and local ground conditions. The formulae can have different forms altogether or have differences in the value of constants and exponents. A literature search and review of earlier experience from the same geographic area is therefore recommended.

These prediction methods could be assumed sufficiently accurate at an early stage of a project or in projects where ground-borne noise and vibration is not particularly severe or not critically sensitive to the ground properties. Geotechnical and rock mechanical parameters entering into the formulae in the following subclauses are more thoroughly defined and explained in textbooks like Reference [16].

6.3.2 Effective (bulk) mass density

If the effective (bulk) mass density ρ of a partly or fully saturated soil is not directly available for the geotechnical in-situ or laboratory measurements, it can be estimated from Formula (24):

$$\rho = (1-\varphi)\,\rho_{\text{mineral}} + \varphi S_{\text{r}} \rho_{\text{water}} \tag{24}$$

where

φ is the porosity of the soil;

S_{r} is the degree of water saturation;

ρ_{mineral} is the mass density of the mineral making up the particles, which for quartz mineral, is close to 2 700 kg/m^3;

ρ_{water} is the mass density of water, which is close to 1 000 kg/m^3.

6.3.3 Wave speeds and elastic shear modulus

6.3.3.1 Soils

In assessing and controlling ground-borne noise and vibration from rail systems, the shear wave speed V_{s} and the corresponding low-strain shear modulus G_{max} are usually considered the most important dynamic ground parameters. The two are related through Formula (2).

G_{max} and thus the shear wave speed depends on the soil characteristics and the effective stress in the soil. If the shear wave speed or G_{max} has not been directly measured in-situ or in the laboratory, it may be estimated from in-situ stress condition and ground material index parameters.

For cohesionless ground materials like sand, coarse silt, ballast, crushed rock, gravel and rock fill, G_{max} can reasonably well be estimated from Formula (25):[46]

$$G_{\text{max}} = B\, f(e)\, p_{\text{a}} \left(\frac{\sigma'_{\text{mean}}}{p_{\text{a}}} \right)^{n} \tag{25}$$

where

B is a dimensionless constant, usually close to 700 for most sands and gravels;

$f(e)$ is a dimensionless function of the void ratio e of the soil:[46]

$$f(e) = 1/\left(0,3 + 0,7e^2\right) \tag{26}$$

The void ratio e is an alternative measure of porosity φ. The two are related through $e = \varphi/(1-\varphi)$.

The application of Formulae (25) and (26) is limited to the void ratio range $e < 1,2$, corresponding to a porosity $\varphi < 0,55$ (i.e. 55 %).

p_{a} 100 kPa is a reference pressure;

σ'_{mean} is the mean effective confining stress in the ground at the depth of the G_{max} determination, $\sigma'_{mean} = (\sigma'_1 + \sigma'_3)/2$ with σ'_1 being the effective normal stress in the direction of the shear wave propagation and σ'_3 the effective normal stress in the direction of particle motion of the shear wave, i.e. the normal stresses in the plane of shear wave polarization. σ'_{mean} can sometimes be approximated by the mean of all three principle normal stresses, i.e. the octahedral effective stress σ'_{oct} in the ground at the depth of the G_{max} determination. More recent elaboration of this class of empirical relation is found in References [44] and [47].

n is the stress exponent which is close to 0,5 for most cohesionless materials.

In reasonably level ground, the vertical effective normal stress σ'_v is determined by the weight of the overburden (submerged weight under the ground water table) and can be considered to be a principal stress. See Formula (24) for estimation of the mass density.

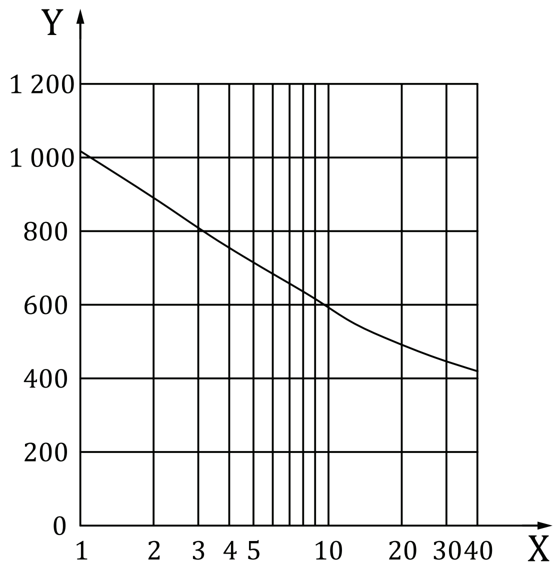
The corresponding horizontal effective normal stress is $\sigma'_h = K'_0\, \sigma'_v$ with K'_0 being the coefficient of effective lateral stress, which can be expected to be in the range between 0,5 and 0,7 for most sites. However, there might locally be geological reasons for higher values.

For cohesive soils like clay and fine silt, G_{max} can be estimated from the undrained shear strength s_u and the plasticity index I_p:[32]

$$G_{max} = \left(20\,800 / I_p + 250\right) s_u \tag{27}$$

NOTE This formula assumes the plasticity index I_p given in %.

Figure 6 presents an alternative estimator for G_{max} which utilizes the over-consolidation ratio OCR as the index parameter for the clay.[26] Figure 6 plots G_{max} / s_u^{DSS} versus OCR. Here s_u^{DSS} is the undrained shear strength as measured in the direct simple shear device DSS. All terms are defined and further explained in References [16] and [26]. Supplementary formulae on this issue are found in References [98] and [99].



Key

X OCR

Y $G_{max}/\ s_u^{DSS}$

Figure 6 — Empirical relation for G_{max} normalized with respect to undrained shear strength s_u^{DSS} for clays, plotted versus over-consolidation ratio OCR

If the dilatational wave speed, V_p, has not been directly measured at the site, e.g. by conventional refraction seismic investigation, it may be estimated from the shear wave speed, V_s. For dry or partly saturated soils, Formula (4) applies for this estimation.

For the dry and partly saturated soils, the value of the low-strain Poisson's ratio of the grain matrix, ν_0, is usually in the range 0,2 to 0,3. In this range, the V_p/V_s ratio is not particularly sensitive to ν_0 and the above range does not introduce more than about ±7 % uncertainty in the V_p estimate. This holds true for degrees of saturation up to about 0,99.

For fully saturated soft soils (typically V_s less than about 200 m/s), the dilatational wave speed, V_p, can roughly be estimated from Formula (28):

$$V_p = \sqrt{\frac{\rho_{water}}{\varphi\left[\left(1-\varphi\right)\ \rho_{mineral}+\varphi\rho_{water}\right]}}V_{water} \tag{28}$$

where V_{water} is the sound speed in water, $V_{water} \approx 1\ 500$ m/s.

Formula (28) is only valid for really high degrees of saturation (>0,999 9). For intermediate saturation and for stiffer soils, the relation becomes more complicated and a more complete Biot-Gassmann fluid substitution scheme needs to be applied.[39] A further discussion on wave propagation in water-saturated granular materials is found in References [40] to [42] and [108].

6.3.3.2 Rocks

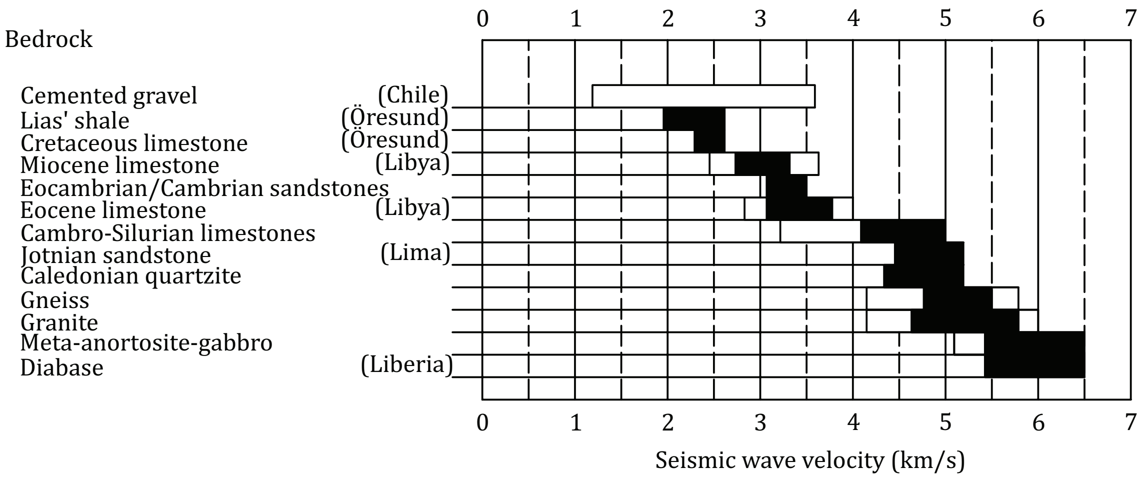
Wave speeds and wave attenuation in rocks is a wide topic, discussed thoroughly in the literature like in References [38] and [39]. One commonly used index parameter for rock property is the rock mass quality value Q_c as defined in Reference [43] and further developed in Reference [38]. Q_c is a dimensionless parameter, determined during the engineering geological classification of a rock mass from parameters like relative block size, relative friction strength of joints, and relative effects of water, swelling, etc. in addition to overburden stress. The Q_c value shall, however, not be mixed up with the seismic quality factor Q defined in Formula (18).

If Q_c values for the rock at the site are available from the engineering geological investigation of a rail system project, the dilatational wave speed, V_p, of the rock mass can roughly be estimated from Formula (29) in units of km/s:[38]

$$V_p = \lg Q_c + 3{,}5 \tag{29}$$

Figure 7 plots typical range of V_p values for various rock types, taken from Reference [43] and also presented in Reference [38].

For most rocks, the value of the low-strain Poisson's ratio ν_0 is typically in the range 0,2 to 0,3 and Formula (4) can be used to estimate V_s from V_p. In this Poisson's ratio range, the V_p/V_s ratio is not particularly sensitive to ν_0 and the range between 0,2 to 0,3 does not introduce significant uncertainty in the V_s estimate.



NOTE Black bars indicate most common range of speed variation. White bars give the likely total range of variation.

Figure 7 — Typical range of p-wave speed, V_p, for little weathered and moderately fractured rocks

6.3.4 Non-linearity and material loss factor

Dynamic strains in the ground related to ground-borne noise and vibration from rail systems are usually very low and in a range where ground materials have a nearly ideal linear elastic behaviour. However, even at these small strains the materials have a small amount of internal loss η_{min}. Towards higher strains (like close to the vibration source, for trans-Rayleigh speeds, etc.), ground materials exert an increasing hysteretic non-linear behaviour, which leads to additional hysteretic loss. Figure 3 illustrates a hysteretic shear stress-strain loop, and defines the initial G_{max} and the secant G_{sec} dynamic shear moduli, and the hysteretic loss factor η as defined by Formula (17).

A convenient way to quantify the dynamic non-linearity of ground material is by plotting the normalized shear modulus G_{sec}/G_{max} versus the cyclic shear strain γ_c as done in Figure 8 a).[47] This G_{sec}/G_{max} curve is for sand, but it is remarkable that mostly any granular, non-cohesive and even low-plasticity cohesive material seems to fit closely into the same curve. In this normalization, the dynamic (cyclic) behaviour of ground materials falls into three regimes, depending on the range of cyclic shear strain, see References [29], [30], [45] and [47]:

a) "very small strains" where the material has a practically linear elastic behaviour, with no irreversible degradation due to the cyclic loading;

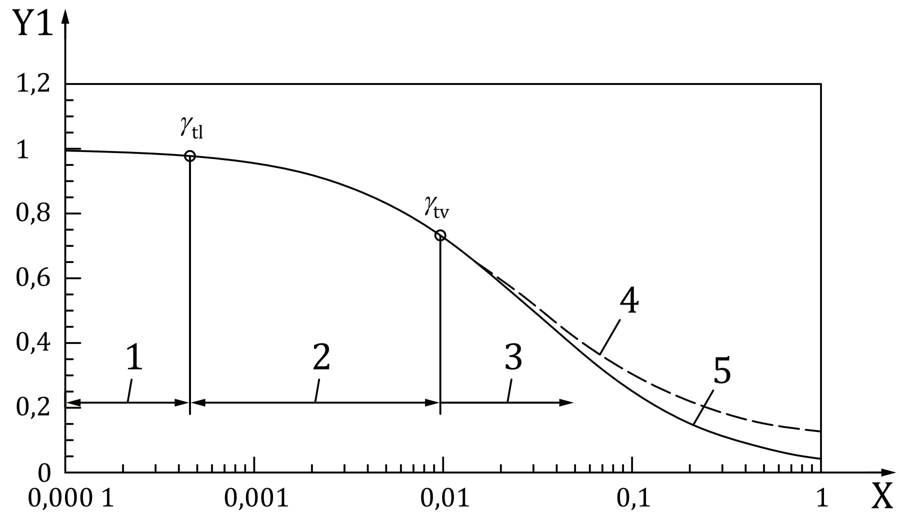
b) "small strains" where the material has a hysteretic non-linear behaviour but still do not degrade;

c) "medium to large strains" where the material has a strong hysteretic non-linearity and where the material gradually degrades with increasing number of cycles.

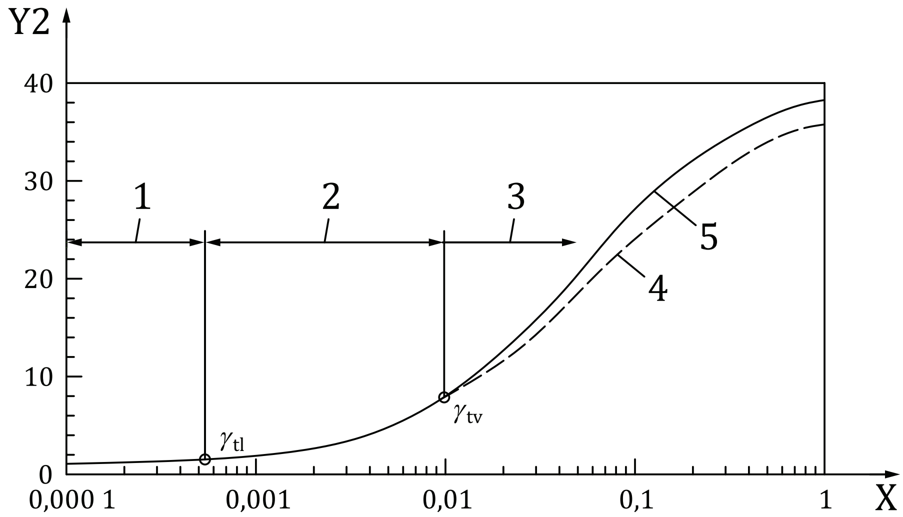
For undrained loading in regime c), the degradation leads to gradual pore pressure build-up and reduction in G_{max} and G_{sec} for increasing number of cycles. However, the G_{sec}/G_{max} ratio turns out to stay reasonably constant making the G_{sec}/G_{max} curve also fit well for degraded material. For drained loading, the material compacts as a consequence of the degradation, leading to increased G_{max} and G_{sec}. Even for this case, G_{sec}/G_{max} stays reasonably unaffected and the curve is still valid.

The strains forming the transition between the regimes are termed threshold strains. The linear cyclic threshold shear strain γ_{tl} forms the border between "very small strains" a) and "small strains" b), and the volumetric threshold shear strain γ_{tv} forms the border between "small strains" b) and "medium to large strains" c). For granular, non-cohesive and low-plasticity cohesive materials, $\gamma_{tl} \approx 5 \times 10^{-3}\ \%$ and $\gamma_{tv} \approx 10^{-2}\ \%$.

Figure 8 b) plots the corresponding hysteretic material loss factor η versus the cyclic shear strain γ_c, divided into the same three strain level regimes.[47] This curve also turns out to fit reasonably well for all granular, non-cohesive and even low-plasticity cohesive materials, non-degraded, as well as degraded.



a) Normalized shear modulus reduction curve



b) Material damping variation curve

Key

1 linear range

2 non-linear elastic range

3 non-linear range

4 N = 1 000 cycles

5 N = 1 cycle

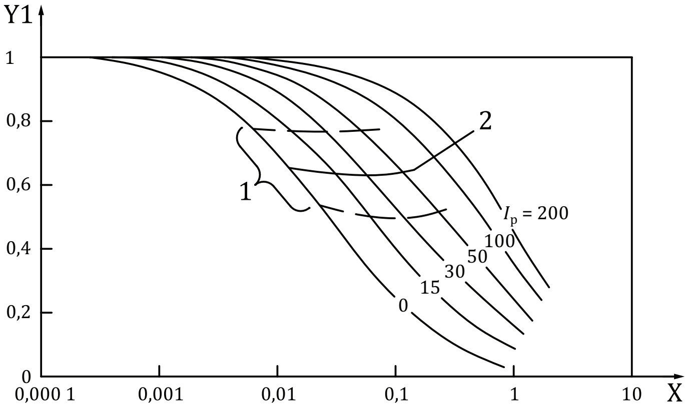
X cyclic shear strain γ_c in %

Y1 normalized shear modulus G_{sec}/G_{max}

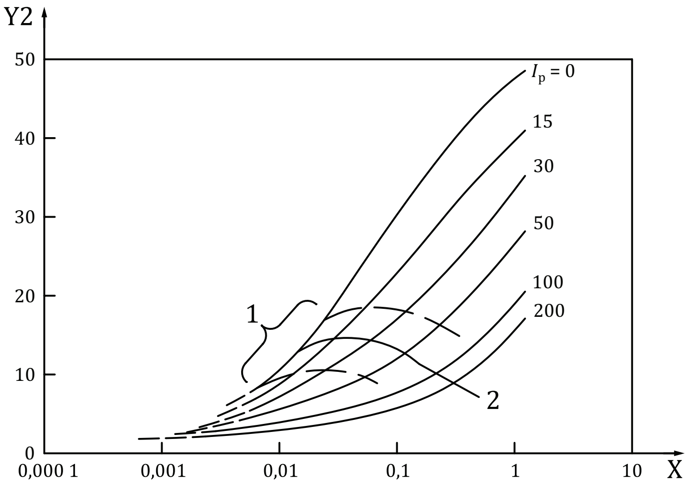
Y2 material loss factor η in %

Figure 8 — Typical non-linear modulus reduction and damping variation curve for cohesionless soil materials

For more plastic cohesive soils (clays), the degree of plasticity turns out to have an effect on the normalized shear modulus G_{sec}/G_{max} and the loss factor η versus the cyclic shear strain γ_c. Figure 9 plots the normalized shear modulus and damping curves for cohesive soils like clay with different plasticity indexes I_p.[29] The curves for I_p = 1 are identical to those for non-cohesive soil materials in Figure 8. For plastic clays also the threshold strains and thus the cyclic behaviour regions depend on the plasticity index.



a) Normalized shear modulus reduction curve
(for N = 1 and OCR = 1 to 15 for a range of plasticity index values, I_p, from 0 to 200)



b) Material damping variation curve
(for N = 1 and OCR = 1 to 8 for a range of plasticity index values, I_p, from 0 to 200)

Key

1 range of γ_{tv}

2 average γ_{tv}

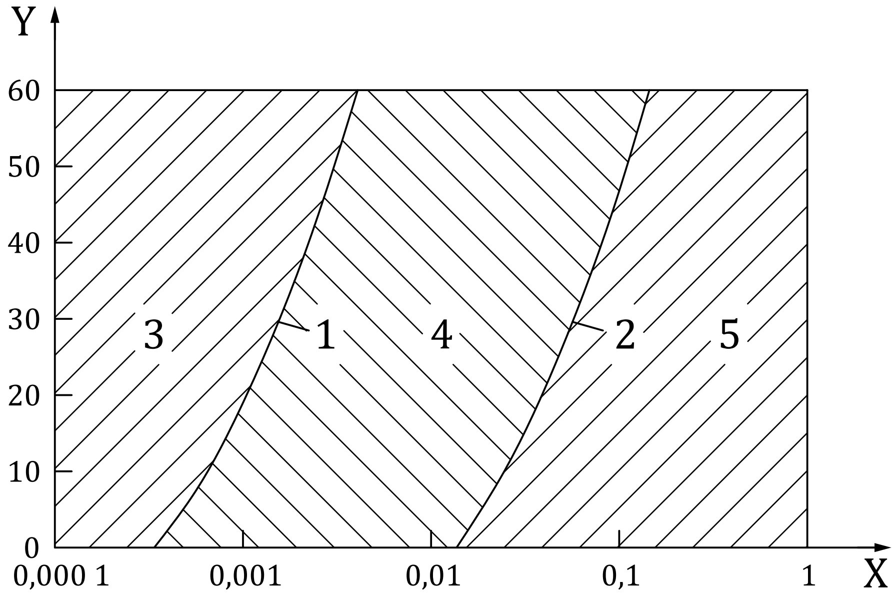
X cyclic shear strain γ_c in %

Y1 normalized shear modulus G_{sec}/G_{max}

Y2 material loss factor η in %

Figure 9 — Typical non-linear modulus reduction and damping variation curves for undrained plastic soils versus plasticity index I_p

Figure 10 plots γ_{tl} and γ_{tv} versus I_p.[29] More information on damping of soil materials is found in References [45] and [47].



Key

1 average γ_{tl} line
2 average γ_{tv} line
3 very small strains
4 small strains
5 medium to large strains
X cyclic shear strain γ_c in %
Y plasticity index I_p in %

Figure 10 — Categorization of cyclic shear strains with respect to response to cyclic loads — linear and volumetric threshold strains versus plasticity index

For materials strictly following the Masing's rule for non-linear hysteresis, there should be a unique relation between the curvature of the $G_{sec}/G_{max}(\gamma_c)$ curve and the $\eta(\gamma_c)$ curve as further explained in Reference [28]. For the flat part of $G_{sec}/G_{max}(\gamma_c)$ in the "very small strain" regime, there is no curvature and conventional hysteresis theory gives zero loss. Reality does, however, show that ground materials exert a small fraction of hysteretic-like loss also in the "very small strain" regime. This loss factor is termed η_{min}. This loss mechanism is important for most issues of ground-borne noise and vibration from rail systems since the cyclic strains are in this range. Typically, the low-strain loss factor η_{min} is 6 % to 14 % for sands and gravels and 4 % to 10 % for clays. For ground-borne noise and vibration from rail systems comes an apparent additional loss contribution which is caused by fuzzy transitions between ground layers, ground inhomogeneities, etc., as further discussed in 5.4. When including this effect, total apparent loss factors when simulating wave propagation from rail systems in real ground can end up as high as 15 % to 20 %.

6.4 Indirect determination from geotechnical in-situ penetration tests

6.4.1 General

Data collected from geotechnical in-situ measurements like cone penetration test (CPT) and standard penetration test (SPT) may be utilized to give rough estimates of G_{max} and thus V_s through empirical correlations.

6.4.2 Cone penetration test

Cone penetration test (CPT) is a frequently used method for in-situ determination of geotechnical parameters at soft to medium stiff ground conditions. The method is based on a standardized tipped cone being continuously pressed into the ground while the tip resistance, the pore pressure and a sleeve friction just above the tip is measured. The method is further described in References [11], [17], [36] and [93]. The key parameter coming out of the CPT test, which can be used to estimate dynamic properties of the ground is the cone resistance corrected for the effect of pore pressure, versus penetration depth $q_t(z)$ measured in kPa.

If the corresponding depth variation of the void ratio $e(z)$ is known, G_{max} can roughly be estimated for cohesive soils like clay from Formula (30):

$$G_{max}(z) = p_a \frac{99{,}5}{e(z)^{1{,}13}} \left(\frac{q_t(z)}{p_a} \right)^{0{,}695} \tag{30}$$

Formula (30) is a reformulation to SI units, based on References [20], [36] and [48]. The reference pressure p_a = 100 kPa is as that one used in Formula (25). G_{max} resulting from Formula (30) has the unit kPa.

For cohesionless soils like sand, G_{max} can be roughly estimated from measured cone tip resistance versus depth $q_t(z)$ according to Formula (31):

$$G_{max}(z) = 1\,634\, q_t(z)^{0{,}250} \sigma_v'(z)^{0{,}375} \tag{31}$$

$\sigma'_v(z)$ is the effective vertical stress in the ground versus penetrated depth. Be aware that G_{max}, q_t and σ'_v all need to have the unit kPa to make the prediction turn out correctly.

NOTE Formula (31) originates from Reference [73].

The two above formulae may give reasonable estimates if applied correctly to clear cohesive or cohesionless soils. If applied incorrectly or applied for mixed soil types, they may yield severely incorrect results. There are however more reliable procedures to estimate dynamic shear modulus or shear wave speed from CPT soundings, e.g. like the one described in Reference [104]. These constitute rather lengthy workflows, where some of the initial steps do classify the soil type. This makes the procedure fare more versatile, reliable and generally applicable than the simpler formulae.

6.4.3 Standard penetration test

Standard penetration test (SPT) is by far the oldest and most commonly used in-situ geotechnical investigation method for medium to hard granular ground. The method is based on a split-barrel sampler being driven into the soil at the bottom of a bore hole. At each interval, the sampler is usually driven 46 cm down and the number of blows required to achieve the last 30 cm of penetration is taken as the standard penetration resistance N. This value is correct for the actual blow efficiency of the applied device and for the effect of the overburden to form the normalized value N_{60}. The sampler, the driving device and the blow rate is to some extent standardized, but there are also a lot of local or national variations. SPT N_{60} should therefore be considered a rather crude measure of the ground resistance. The method is further described in ISO 22476 and Reference [17].

Where there is lack of better information, the SPT N-value may, however, be used to give indicative predictions for the dynamic properties of the ground through the following empirical formulae. They are given as examples, although other relationships exist and could be utilized.

$$V_s(z) = a N_{60}(z)^b \tag{32}$$

where

$N_{60}(z)$ is the normalized blow-count versus depth z from SPT sounding;

a and b are coefficients, depending on the local soil type.

Typical values for a are in the range 80 to 120; b is close to 1/3. Some sets of coefficients for specific soils are proposed in References [33], [35], [100] and [101]. The resulting shear wave speed is expressed in m/s.

A formula expressing the relationship between N value and G_{max} is:

$$G_{max}(z) = 4{,}38 N_{60}(z)^{0{,}333} \sigma'_{mean}(z)^{0{,}5} \tag{33}$$

Formula (33) predicts G_{max} in MPa while $\sigma'_{mean}(z)$ is the corresponding mean effective overburden pressure versus depth, given in kPa. Formula (33) is based on References [20] and [34] but converted to SI units. The predictions are most reliable for coarse-grained soils and less reliable for fine-grained soils.

The indirect estimates of dynamic ground parameters from geotechnical drilling results, described in 6.4, are always uncertain and should only be taken as rough estimates. They can be used as screening methods at an early stage of a project. If screening indicates that groundborne sound and vibration may become an issue, direct in-situ or laboratory measurements of the dynamic properties of the ground, as described in 6.5 and 6.6, are far more reliable and are recommended at the further stages of the project. Uncertainty should be accounted for by assuming a reasonable range of variation in the determined parameters, by randomly sampling the variation among the variables and performing sensitivity studies.

6.5 Direct in-situ measurement of dynamic ground parameters

6.5.1 General

Far more reliable data on the dynamic properties of the ground can be obtained by direct in-situ measurement at the site, than what can be achieved by the more indirect methods described above. Such direct measurement methods utilize wave propagation in various forms and are termed seismic methods. Those methods particularly suited for ground-borne noise and vibration from rail systems do primarily focus on the shear wave speed of the ground. However, the methods can also measure the p-wave speed and to some extent the loss factor.

The following subclauses present in more detail: Surface wave methods, down-hole seismic CPT (S-CPT) methods and cross-hole methods. Some other seismic geophysical methods are briefly summarized.

For rail system ground-borne noise and vibration issues, the non-intrusive surface wave methods are for most cases the best suited and most cost effective to measure reliably the shear wave speed parameters for the ground. Particularly for soft soil sites, the seismic CPT variant of down-hole measurement is an alternative, or more often a supplement for more detailed measurements at local positions. Other variants of the down-hole, up-hole and cross-hole methods are also presented as they can be preferred under particular conditions.

Table 1 gives an overview of all presented methods, and summarizes their main characteristics, advantages and disadvantages. A more general overview of the applicability of various geophysical methods is found in Reference [96].

When dynamic ground parameters determined from in-situ measurements are to be utilized in noise and vibration prediction and design calculations, it is important to keep in mind that these parameters largely depend on the effective confining stress in the ground. The parameters are therefore only valid for the in-situ stress situation where they have been measured. If the parameters are to be used for another confining stress situation, e.g. after an excavation or an in-infill of masses has been performed, the change in stress needs to be corrected for.

Table 1 — Direct measurement of dynamic ground parameters — Summary and evaluation of in-situ methods and laboratory methods most suited for rail system ground-borne noise and vibration

Method	Key features	Advantages	Disadvantages	Suited for determination of						
				Layering	V_s or G_{max}	V_p or M_{max}	ν_0, S_R	Low strain η_{max}	G_{sec}/G_{max}	High strain η
In-situ methods										
Surface wave methods	The speed of surface waves along an instrumented linear profile is measured versus frequency (dispersion). This dispersion information is inverted to give s-wave speed versus depth for the profile. Practical penetration 30 m to 50 m.	Well suited, efficient, commonly used. Do not require any drilling. Sensors placed on ground surface. Source on ground surface. Works on any ground. Gives averaged properties along profile. Gives high resolution close to the surface.	Limited penetration. Deep penetration requires hard to achieve low-frequency wave source. Limited resolution at larger depth. Do not or do hardly pick lateral variability in ground properties along profile. Lateral variability can reduce quality in results.	2	3	1	1	1	0	0
Seismic CPT (S-CPT) (variation of down-hole)	Travel time for s-waves measured from a source at the surface to one or more vibration sensor mounted in a CPT tip. Consecutive penetration and measurement produce s-wave versus depth. Works also for p-waves. Practical penetration 30 m to 60 m in soft soil. Depth resolution 0,5 m to 1 m.	Commonly used. High accuracy and resolution but slightly lower than cross-hole. Easy and fast penetration operation during measurement. Provides ordinary CPT data from same run.	Can not penetrate hard ground and ground with gravel and boulders. Depends on reliable trigger signal if only one sensor.	3	3	2	2	1	0	0
Down-hole suspension logger	Travel time for s-waves (and p-waves) along a borehole measured from source to receivers, both contained in a logging tool lowered into a predrilled hole filled with water or (drilling fluid). Clamping needed for s-waves. Penetration depth unlimited as deep as drilling practically possible. Depth resolution 0,5 m to 1 m.	New technology. Limited experience. Still high accuracy and resolution but slightly lower than cross-hole. Does not need more than one predrilled hole. Can log additional parameters.	Depends on reliable trigger signal. Logging tool-borehole interaction can obscure measurements.	2	2	3	2	0	0	0
Cross-hole	Travel time for s-waves and p-waves measured from source clamped in one predrilled hole to receivers clamped in one or more parallel holes. Penetration 20 m without deviation measurement; unlimited with deviation measurement, as deep is drilling is possible. Depth resolution 0,5 m to 1 m.	Accurate wave speed measurement and high resolution in resolving layers. Works for soils and rock. Can enable tomographic measurements.	Requires drilling (and casing) of two or more holes. Relies on parallel holes. Vulnerable to nearfield effects. Large stiffness contrasts can cause inaccuracy due to refraction.	3	3	3	3	1	0	0

NOTE Rating: 3 excellent, 2 reasonable, 1 marginal, 0 not applicable.

Table 1 *(continued)*

Method	Key features	Advantages	Disadvantages	Suited for determination of						
				Layering	V_s or G_{max}	V_p or M_{max}	ν_0, S_R	Low strain η_{max}	G_{sec}/G_{max}	High strain η
Laboratory methods										
Piezo bender	Shear waves are generated by a piezo bender element, transmitted through the specimen and received by another piezo bender at the other end. Wave speed determined from recorded travel time and sample height. Piezo benders are ideal for generating and recording s-waves, and have an ideal impedance match to soft to medium stiff soils. To also measure p-waves, an additional set of solid (not bender) piezo crystals are needed. For this reason, the rating is given in brackets.	Works within ordinary triaxial and DSS test devices. Measurements can be made at any stage of a test. Fast test, can make many repetitions.	Strain level is low and cannot be controlled.	0	3	(3)	(3)	0	0	0
Resonant column	A cylindrical specimen is tuned into resonance. From resonance frequency, response wave speed and material loss factor can be determined.	Strain level can be controlled. Tests can be made at different cyclic strain (stress levels). Works from very low to moderate strains. Linear elastic to non-linear elastic. One type can measure at various stages during a triaxial test.	Most applicable for soft soils.	0	3	0	0	3	3	3
Cyclic triax, DSS	Samples in triaxial of DSS test mode are exposed to cyclic (repeated) loading and the response is measured. Complete stress-strain loops are recorded. Secant modulus and material loss factor can be determined.	Can go to high strains (stresses). Covers the non-linear elastic and non-linear regimes.	Test performed at low frequency. Not reliable for the linear elastic range.	0	1	1	1	1	3	3
NOTE Rating: 3 excellent, 2 reasonable, 1 marginal, 0 not applicable.										

6.5.2 Surface wave measurements

Seismic surface wave measurement is a non-intrusive method to map the dynamic in-situ properties of the ground locally and over larger sites. Since the method is non-intrusive, no drilling is needed and it works well for the softest grounds to hard grounds and soils containing gravel and boulders where devices like seismic CPT (S-CPT) cannot penetrate and where even drilling is difficult. Surface wave methods provide averaged dynamic properties over significant volumes of ground material, as opposed to more “needle stitch” values obtained by the down-hole, S-CPT, laboratory and partly by the cross-hole methods. Averaged properties are most representative and what is mostly in demand when dealing with ground-borne noise and vibration issues related to rail system projects.

High-capacity, low-cost computers, as well as advances in signal processing and numerical modelling of wave propagation, has opened new possibilities within the area of seismic surface wave measurement. A multitude of variants of the method and data processing routines are available. This subclause, however, is limited to provide a general description and give recommendations for application in ground-borne noise and vibration from rail systems.

The principle of the surface wave methods relies on the basic properties of seismic waves, and particularly Rayleigh type surface waves, as described in 5.4. These waves propagate along the surface and their associated particle motion is confined to a layer roughly penetrating half to one wavelength into the ground. A high-frequency wave component with a short wavelength will therefore “sense” only the wave propagation properties of the upper part of the ground, whereas a low-frequency component, due to its longer wavelength, will sense the properties down to a larger depth. This leads to a mix-determined situation in which not all frequency components sample the same part of the subsurface. As the body wave speed, and s-wave speed in particular, in the ground varies with depth, the wave speed of the surface waves becomes frequency- (or wavelength-) dependent. In other words, surface waves are geometrically dispersive and their dispersion characteristics reflect how the wave speed in the ground varies with depth. The speed of Rayleigh type surface waves is closely linked to the speed of shear body waves (s-waves) in the ground. The surface wave dispersion therefore mostly reflects how the s-wave speed of the ground material varies with depth. Surface wave measurement is therefore primarily a method to measure the shear wave speed versus depth of the ground materials at the site.

Surface wave measurements consist of three basic steps (see Reference [78]):

a) plan and design the surface wave field measurement and record their propagation along the profile to be investigated using a proper seismic source and receiver array (sufficient energy, wide-band frequency content, phase-calibrated receivers, consider nearfield effects, etc.);

b) process the recorded seismic data to deduce the surface wave dispersion characteristics;

c) use the dispersion data (i.e. phase or group velocity or slowness as a function of frequency or wavelength) in a tuned forward modelling or inversion process to deduce shear wave speed and other dynamic characteristics of the ground in the investigated profile, as a function of depth.

The various implementations of the method deviate in the way the waves are generated, how the sensors are laid out along the profile, the type of sensors used, how the dispersion is determined and how the inversion is performed; see Table 2.

In layered ground, surface waves propagate as several modes. Some of the implementations are able to detect not only the fundamental, but also higher modes and utilize their dispersion in the inversion process. The primary outcome of the inversion is for all implementations the shear wave speed versus depth, most often as a 1D profile and occasionally as a 2D profile. To various degrees, the implementations also deduce p-wave speed and wave attenuation properties versus depth. These parameters are, however, estimated with far less accuracy compared to the shear wave speed and layer thickness.

Table 2 — Surface wave method for measurement of dynamic ground properties — Summary and evaluation of variations of the method

Variations		Key features	Advantages	Disadvantages
A — Multi-station approaches, multi-channel analysis of surface waves (MASW)				
a	2D FFT, *f-k* transform	Fourier transform data from time-distance into frequency-wave-number domain. Determines dispersion curves and phase velocity based on proportionality to ratio between frequency and angular wave number.	Can determine higher modes in addition to fundamental mode. Method has high resolution. If a larger number of sensors is used sub-grouping enables determination of lateral variation along profile.	Requires a large number of simultaneously sampled stations, at least 24 to 48, preferably more (up to 256), spaced 0,5 m to 1 m. Requires regular sampling in time and space. Vulnerable to spatial aliasing due to under-sampling in space.
b	Phase slowness time intercept (τ-p transform)	Applies a slant-stack of the common-shot wave field from time-distance domain into phase slowness time intercept (reduced time t) domain, followed by a Fourier transform on reduced time axis.	Same as for A – a. Regular sensor spacing not required.	Large number of sensors stations required, densely spaced. Demanding data processing.
c	Phase shift method	Data Fourier transformed into frequency-distance. Phase shift is applied according to range before stacking for each frequency.	Insensitive to data processing. Regular sensor spacing not required.	Large number of sensors stations required, densely spaced. Demanding data processing. Sensitive to resolution in slowness
d	Frequency decomposition slant-stacking	Impulsive data stretched into frequency-swept data. Slant-stacked for range of velocities for each frequency to remap data to phase-velocity versus frequency.	Improves resolution and mode separation. Regular sensor spacing not required.	Large number of sensors stations required, densely spaced. Demanding data processing.
e	High-resolution Radon transform	Time-distance domain data transformed into frequency-distance. High-resolution linear Radon transform applied to each frequency.	Regular sensor spacing not required. Gives improved resolution, also at low frequency and deep penetration.	Large number of sensors stations required, densely spaced. Demanding data processing.
B — Limited-station approaches				
a	Time-frequency representation (TFR)	Multiple filter analyses applied to selected traces. Apply wavelet transforms or Gabor transforms. Determines group velocity versus frequency.	Works on single or a few measurement stations. Can pick lateral variability	Limited resolution.
b	Spectral analysis of surface waves (SASW)	Successively moves a set of sensors (2 or 4) from small to larger distances for a common midpoint excitation. Phase velocity versus frequency determined from phase difference between arrival at different stations	Small number of measurement stations and limited number of sensors and measurement channels needed.	Largely relies on successful phase unwrapping. Long acquisition time.
c	Continuous surface waves (CSW)	Uses a continuous vibratory source that runs at one frequency at a time. Sensors are moved along ground to locate nodal points and thus wave length versus frequency. Alternatively, a receiver array is used and wavelength determined from slope of phase angle versus distance.	Has a high signal to noise ratio. Insensitive to background disturbance. Few sensors and measurement channels needed.	Long acquisition time. Limited penetration depth unless heavy shaker is applied.
C — Passive surface wave methods				
a	Passive array method	Utilize ambient vibration. Utilize directional coherence to separate wave packages.	No source needed. Can give deep penetration.	Long acquisition time. 2D sensor array needed. Works better offshore than on land.

For energy and frequency bandwidth considerations, recommended practice is to use various types of seismic sources (e.g. drop weights and other vertical impact sources) to generate the surface waves. The challenge is to generate in total sufficient wave energy over the whole frequency range needed to cover the requested investigation depth range. Recorded wave data from various sources need to be combined to build the complete picture. Stacking shall be done with care to account for differences in phase behaviour among the various sources. It is particularly challenging to produce enough energy at sufficiently low frequency to obtain deep penetration. Practically obtainable maximum investigation depth at soft to medium stiff ground is about 30 m to 50 m.

Primary sensors should be vertical and well coupled to the ground surface. Some additional horizontal sensors can help in the inversion. Either accelerometers or geophones can be used. However, vitally important is sufficiently low-frequency response to cover the requested maximum penetration, and that the phase matching among the sensors is accurate (with calibration checks). Ideally, as many sensors as possible should be used. Good practice for a reliable investigation is to use at least 48 to 96 primary sensors equally spaced at 0,5 m to 1 m along a straight line over the profile to be investigated. The measurements may be performed by using a lower number of sensors, and repeating the measurements after having consecutively moved the sensors. However, other survey designs can be used, e.g. with irregular receiver spacing, or using only few receivers. Both spectral analysis of surface waves (SASW)[82] and multi-channel analysis of surface waves (MASW)[78] can then be performed on the same data sets. Note that the length of the instrumented profile should be at least equal to the desired investigation depths. In addition to extending the investigation depths, longer arrays are superior in separating the different surface wave modes; however, the data become more sensitive to lateral variations in soil conditions. Signals from all sensors need to be sampled simultaneously, and with sufficiently high sampling rate.

Whereas there are different ways of processing the surface wave data (see Table 2), all methods essentially rely on mapping phase differences between the different receivers. The most straightforward signal processing to obtain dispersion curves from surface wave data sets is through a two-dimensional Fourier transform of the time records versus offset distance into a frequency versus wavenumber representation. This approach requires equidistant and dense spacing of the sensors along the ground and a reasonably high number of sensors as recommended under “good practice”.

Figure 11 illustrates a typical field measurement setup, the steps in data processing and inversion, the resulting dispersion curves and the best estimate V_S profile, with error indicators.

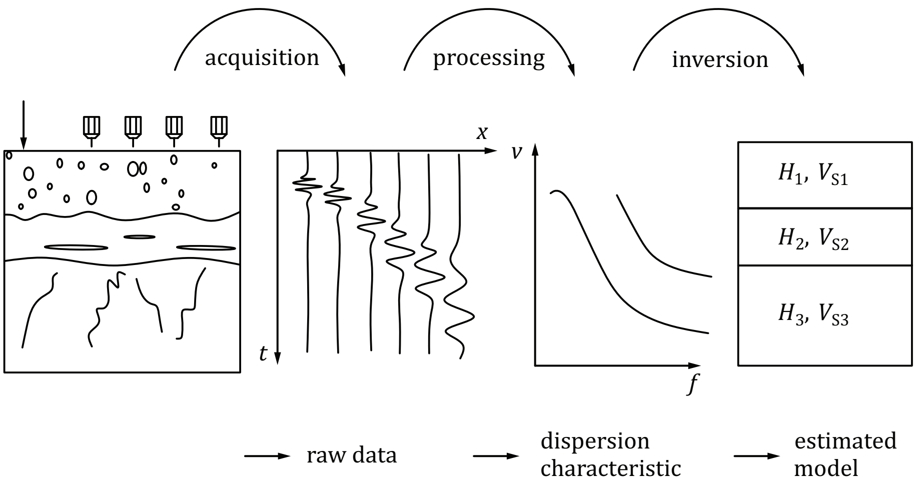


Figure 11 — Surface wave measurement setup and results

Most of the inversion schemes assume the ground to be laterally homogeneous under the extent of the instrumented profile and their results represent averaged properties over this length. Some schemes do have the capability to resolve some lateral variation, however, at the cost of the general determination accuracy. The resulting wave speed versus depth has a high resolution and high accuracy close to the surface, whereas resolution and accuracy deteriorate with depth towards the investigation depth limit. It is therefore important that parameter space, uncertainty and resolution be reported.

In addition to the V_S profile, intrinsic attenuation (e.g. loss factors) can also be assessed from surface wave data, through a coupled analysis and inversion. This is, however, less used.

Supplementary, a priori information about layering, mass density, etc. from the geotechnical site investigation largely contributes to make the surface wave inversion more reliable. A combined survey with some down-hole measurement, preferably S-CPT, and several surface wave investigation profiles to cover the whole site is often an optimum strategy.

Surface wave measurement is also an effective method for more local determination of dynamic properties of ballast, compacted gravel, crushed rock fill, etc. The method is particularly suitable to document the effect of compaction by doing comparative measurements before and after compaction work.

For further information with respect to surface wave data acquisition, processing and inversion, reference is made to References [49] to [87] and [107].

6.5.3 Down-hole (and up-hole) measurements — Seismic CPT (S-CPT)

Down-hole seismic measurements are performed by the use of only one borehole. Typically, a seismic source is placed on the ground surface close to the hole and a seismic sensor is clamped towards the wall in the hole as illustrated in Figure 12 a). The arrival time of the seismic wave is recorded at the sensor depth relative to a trigger signal from the source or alternatively from a sensor close to the source. By moving the receiver one depth interval down (or up) the hole and repeating the arrival time measurement, the average vertical wave speed $\bar{V}$ in the ground material over that depth interval can then be calculated by Formula (34):

$$\bar{V}\left(z_{\mathrm{n}}\right)=\left(z_{\mathrm{n}+1}-z_{\mathrm{n}}\right)/\left(\Delta t_{\mathrm{n}+1}-\Delta t_{\mathrm{n}}\right) \tag{34}$$

where

z_{n} and $z_{\mathrm{n}+1}$	are the depth values of sensor positions n and n + 1, respectively;
Δt_{n} and $\Delta t_{\mathrm{n}+1}$	are the corresponding arrival time readings.

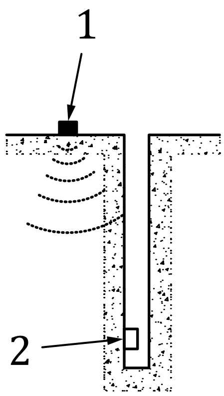
Methods for arrival time picking are the same as described for cross-hole measurements (see 6.5.4). Typical depth intervals between measurements are between 0,5 m and 1 m. By consecutively moving the sensor further down (or up), step by step and repeating the measurements, a profile of wave speed of the ground material versus depth along the hole is established. A standardized procedure for down-hole measurement is found in ISO 22476 and ASTM D 7400. As an alternative to this stepwise interval measurement procedure, a string of equally spaced sensors in the borehole can be used, which allows for a more detailed and accurate analysis of the signals recorded at different depths from an identical source signature.

For measuring s-waves, a horizontally acting source on the ground surface and a horizontal seismic sensor are used. Most common is an impulsive source in the form of a plank or bar pressed against the ground by a dead weight and hit by a hammer at the end. By hitting the other end, a wave with reversed polarity is generated. The wave generated is vertically propagating and horizontally polarized. Alternatively, a vibratory source may be used. If the distance from the source to the borehole is significant compared to the depth of measurement, a correction for the inclined wave travel path is needed. By turning the direction of the source and the receiver 90° around the vertical axis, and repeating the measurement, anisotropy in the dynamic ground properties can be quantified. By using a vertically acting source and a vertical sensor, p-waves can be measured.

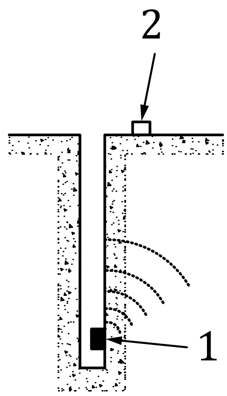
Down-hole tests work both in soils and rocks. In loose soils, the holes need a lining, preferably made of plastic tubing. Under other conditions a backfill material can be needed to ensure proper coupling between the lining and the surrounding soil.

Up-hole measurement is a variation of the same setup, where the seismic source is clamped in the borehole and the sensor is on the ground surface as shown in Figure 12 b). In this setup, a vertically operating source generates p-waves. To generate s-waves, a rotary source, like the one described for cross-hole, is the only option. One-sensor down- or up-hole measurements will lack some accuracy since they rely on a not necessarily repetitive triggering signal.

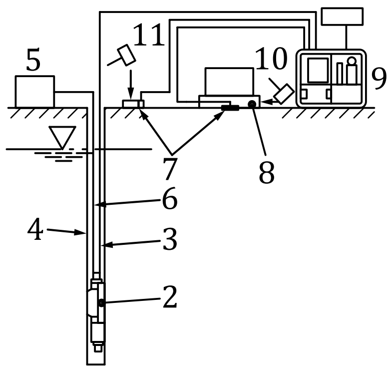
The p-wave measurements both in down-hole and up-hole tests can be unreliable due to unintended interaction with the borehole lining tube.



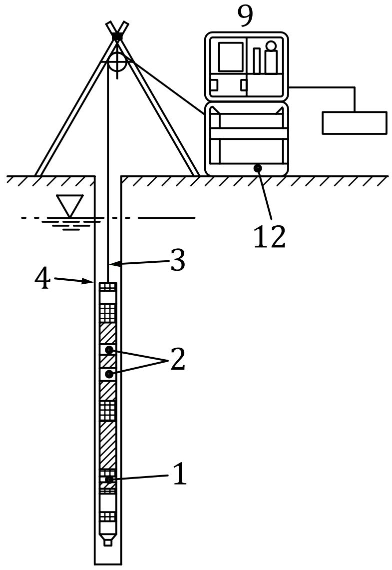
a) Down-hole measurement principle



b) Up-hole measurement principle



c) Down-hole measurement setup



d) Up-hole measurement setup

Key

1 source (exciter)

2 receiver (which for the down-hole measurement setup includes a pneumatic clamping device)

3 cable

4 borehole

5 pressure generator

6 pressure hose

7 excitation detectors

8 plate with static load

9 amplifier, recorder

10 s-wave excitation

11 p-wave excitation

12 cable winch

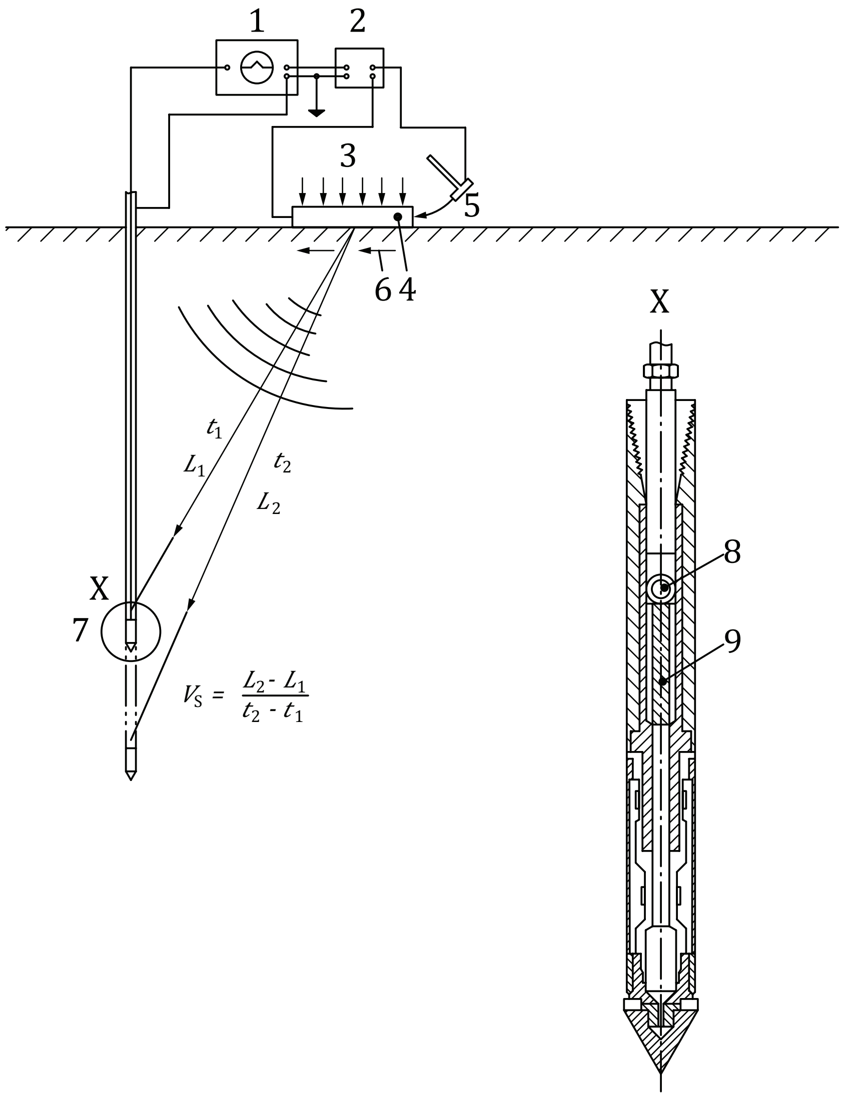
Figure 12 — Down-hole and up-hole measurement setup

The most suited practical implementation of the down-hole measurement is the seismic CPT device (S-CPT). It facilitates efficient down-hole measurements in soft ground, since no predrilled borehole is needed, and is well suited for applications related to ground-borne noise and vibration from rail systems. The equipment consists of a conventional CPT-cone equipped with a seismic sensor in the form of a horizontal geophone or accelerometer mounted within the device just above the friction sleeve as illustrated in Figure 13. More details are found in ASTM D 5778, References [25] and [36]. A slope sensor may also be included for the purpose of tracing deviation for the case the S-CPT is also used for cross-hole measurement, as mentioned in 6.5.4. The measurements follow exactly the same interval procedure as described for conventional down-hole measurements, and can be combined with performing geotechnical CPT sounding in the same run. The cone penetration is halted shortly to make a seismic measurement at each prescribed depth interval; typically at each 0,5 m to 1 m. S-CPT is best suited for soft to medium soils, where a sufficient penetration can usually be obtained. In harder soils and particularly in soils with stones and boulders S-CPT is usually not applicable due to penetration problems.

A high-frequency seismic signal with sufficiently high sampling is desirable for down-hole S-CPT measurements, to obtain sharp arrival time detection and reduce nearfield problems. However, like for cross-hole there is a trade-off due to attenuation of higher frequencies in the ground. For down-hole, it does, however, become more important than for cross-hole, since it directly limits the maximum penetration depth of the measurement. Practically, penetration for down-hole and S-CPT measurements is limited to 30 m to 60 m in soils. S-CPT devices are available with one or with two (or more) sensors. In the two- or multi-sensor devices, the seismic sensors are located in the drill-rod, at typically 1m vertical separation. These devices are superior over those with only one-senor, since they directly measures the interval speed over the sensor separation distance, and do not depend on unreliable trigger signals.

Up-hole and down-hole measurements can also be made by means of a logging tool lowered into predrilled boreholes. The logging tool contains both the wave source and the receivers at certain vertical spacing, as illustrated in Figure 12 d). This technology is adopted from hydrocarbon well logging methods. The logging tools usually measure both p-wave and s-wave speed along the borehole and may also contain sensors for mass density, water content, resistivity, etc. Various versions of this tool are termed “vertical seismic loggers”, “PS suspension loggers” and also “vertical seismic profilers” (VSP). These tools may also utilize the various tube-waves along the borehole and do work in lined boreholes and can give properties of the ground outside the lining. The interpretation requires a numerical model of the dynamics of the tool and the fluid in the borehole, and the lining.

NOTE The vertical seismic profiler is not to be confused with offshore VSP which is an involved seismic method with a multitude of sensors in the borehole, source on the seabed or in the borehole, and alternatively also with sensor cables along the seabed.



Key

1	oscilloscope	6	shear wave source
2	trigger circuit	7	seismic receiver
3	static load	8	seismometer
4	metal beam, wooden beam	9	slope sensor
5	hammer		

NOTE More details of a CPT device are found in ASTM D 5778 and in References [25] and [36].

Figure 13 — Seismic CPT measurement setup (seismic cone penetrometer SCPT)

6.5.4 Cross-hole measurements

Conventional seismic cross-hole testing uses two or more boreholes to measure dynamic properties of the ground for waves propagating between the holes.

In its simplest form cross-hole testing uses two parallel vertical holes, one for an impulsive seismic energy source and one for a seismic receiver. By fixing the source and the receiver at the same elevation in each of the holes, the wave speed in horizontal direction can be determined from the distance between the holes, d, divided by the measured travel time between a trigger signal from the source and the arrival of the wave at the receiver, $\Delta t(z)$:

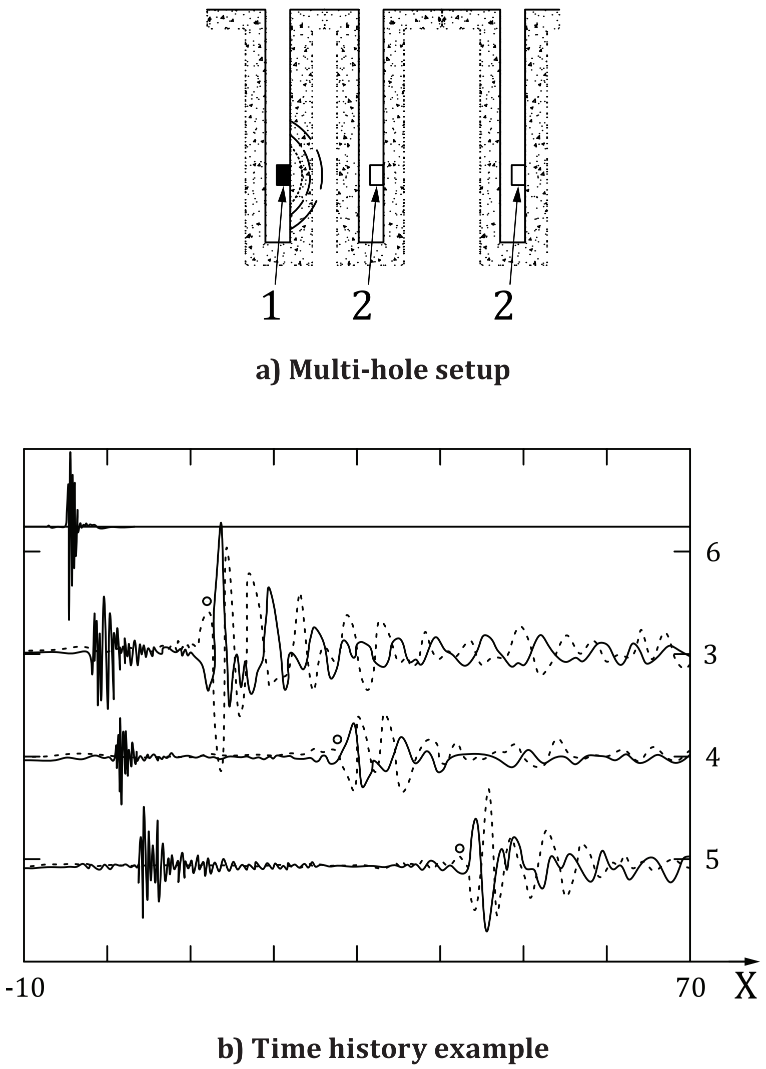
$$V\left(z\right)=d\,/\,\Delta t\left(z\right) \tag{35}$$

By repeating the measurement at consecutive elevations a profile of wave speed versus depth can be established for the ground at the site. Typical depth interval between measurements is from 0,5 m to 1 m.

More reliable and accurate measurements are obtained by using more than two parallel boreholes, see Figure 14 a). Wave propagation speed can then be calculated from the difference in arrival time at adjacent holes. This eliminates errors from uncertainty in the trigger signal, from variability in the coupling of the source and receiver to the ground, and from spherical wave effects close to the source. It also makes interval time picking more precise. Arrival times may be picked by eye using points of common phase in time domain display as shown in Figure 14 b), or by using cross-correlation or cross-spectral (phase angle based) methods. A standardized procedure for cross-hole measurement is found in ASTM D 4428/D 4428M.

Cross-hole tests work both in soils and rocks. In loose soils, the holes might need a lining, preferably made of plastic tubing. A backfill material can also be needed to ensure proper coupling between the lining and the surrounding soil. In soft soils, the source and receiver(s) may as an alternative be consecutively penetrated as the measurements progress downwards, without the need for predrilled holes. Since the determined wave speed is sensitive to uncertainty in the inter-borehole spacing, borehole deviation surveys might be needed for holes deeper than typically 20 m. This depends on ground condition, drilling equipment and requested accuracy.

The cross-hole measurement method can be adapted both for p-wave and s-wave measurements. For pure p-wave surveys, a small explosive source and hydrophone receivers may be used at sites where the boreholes can be kept water filled. Unless, and when s-waves are to be measured, source and receivers need to be clamped towards the borehole wall. Either a mechanical impact or a vibratory source may be used. Such mechanical sources may generate both s- and p-waves. For s-waves, a source working vertically and generating vertically polarized, horizontally propagating waves is most common and easiest to implement. It is, however, possible to implement a rotary source which generates horizontally polarized, horizontally propagating s-waves. By combining both types of sources, anisotropy in the ground material can be measured. To ease the arrival time picking, a source that can reverse its polarity (if impulsive) is beneficial as Figure 14 b) illustrates by solid and dashed lines. Receivers may either be geophones or accelerometers, mounted horizontally in the radial direction for p-wave measurement and vertically, and eventually horizontally in the tangential direction for s-wave measurement. When applying the technique of consecutive penetration, a device like a SPT may be used as the source and a seismic CPT device (see 6.4.2) may be used as receiver(s). This effectively cuts the cost of the investigation as there is no need for predrilling holes.



Key

1	source
2	receivers
3	signal from receiver R1 (The high-frequency first arrival is the p-wave signal, the later, lower-frequency arrival is the shear wave signal)
4	signal from receiver R2
5	signal from receiver R3
6	trigger signal
________	excitation in upward direction
- - - - - -	excitation in downward direction

Figure 14 — Cross-hole measurement

Cross-hole testing can give reliable s-wave results down to typically 50 m to 80 m. For explosive-source p-wave measurements (in rock), deeper investigations are possible. A high-frequency source signal with very high sampling rate is desirable to avoid nearfield effects and to make arrival time histories sharper. The trade-off is due to high attenuation of high-frequency waves. Cross-hole testing is considered an accurate and reliable method for measuring the wave speed in the ground, as long as the inter-borehole spacing is kept under control. The method does, however, rely on the assumption that the ground is horizontally stratified and that the properties do not vary substantially in lateral direction over the distance of the borehole spacing. The method can also give erratic results close to distinct horizontal speed contrasts, as the first arriving waves might not have propagated horizontally, but rather have been refracted into the higher speed layer.

From cross-hole measurements with more than two boreholes, it is possible to determine the attenuation properties, i.e. the loss factor of the ground material in addition to the wave speed. The method does, however, require high-quality seismic coupling between the surrounding ground, the borehole lining and the sensors. The accuracy of the determination is limited since it has to be separated from the geometrical attenuation of the waves between the boreholes, which is influenced by the layering of the ground. Another challenge is to overcome nearfield effects. To resolve loss factor in the nearfield is complex and the deduction should preferably be done outside this zone. However, the nearfield extends typically three wavelengths. A high-frequency source should be preferred to reduce the extent of the nearfield zone.

If the ground properties vary substantially both vertically and laterally, an extended tomographic version of the cross-hole method may be applied. In this approach, the arriving waves from any source position are recorded by a large number of receivers distributed along the receiver borehole(s). By recording the arriving waves from all sensors and for all source positions along the source hole, and subject the recordings to a tomographic inversion, a complete 2D mapping of the wave speed (and attenuation properties) of the ground materials between the boreholes is possible. More information about tomographic measurements is found in Reference [38].

6.5.5 Other measurements — Refraction and multi-channel p- and s-wave reflection, resistivity

A conventional seismic refraction survey is often performed at an initial stage of the geotechnical and engineering geological investigation of a rail system site. However, in this part of ISO 14837 refraction survey is not considered a dedicated measurement method for ground dynamic properties in the context of ground-borne noise and vibration for rail systems and is therefore not further explained here. Description can be found in ASTM D 6429 and in text books like References [18] and [20].

A conventional refraction survey can, however, give useful information about the stratification of the soil, location of ground water table and bedrock at the site. This information is useful as geometric input for building dynamic computational models of the site, but also for planning specific measurements of dynamic ground parameters and can be a support in the inversion of surface wave measurements.

The refraction survey gives p-wave velocities for the ground materials at the site which can be used directly as input in noise and vibration propagation studies. The p-wave speed is an important dynamic property for unsaturated soils and particularly for rocks.

P-wave reflection investigations are usually not performed for mapping the shallow stratigraphy of on-land sites. When used, their purpose is hydrocarbon or mineral exploration, where the target depth can be from several hundred meters to kilometres.

For shear waves, however, recent promising development of multi-channel shear wave reflection survey methods for shallow on-land applications has taken place, using land-streamers and shear-wave vibrators.[70],[71],[76] Shear-wave reflection surveying yields superior results in terms of subsurface imaging, lateral variations and stratification compared to p-wave refraction and surface wave inversion, due to the shorter wavelength of s-waves compared to p-waves. Processing essentially follows the same principles as conventional p-wave reflection surveying.[87] Note that surface waves will typically contaminate the data, except when pure shear is used (i.e. cross-line shear wave vibration direction and cross-line recording direction) from a stiffer interface overlying the softer subsurface (e.g. pavement), which prevents the generation of Love surface waves.

Also geo-electric measurements, like earth resistivity tomography (ERT) can give valuable overview information about soil types, stratigraphy, water table and bed rock location, etc.,[94] but no direct measurement of the dynamic properties.

In essence, it is the combined or integrated use of geophysical and geotechnical techniques that is key to a proper understanding of the soil conditions at a given site.

6.5.6 Other in-situ methods

There are also other in-situ methods that can be useful to measure dynamic ground parameters under certain conditions, like various types of dynamic plate loading tests, dynamic screw plate tests, dynamic vane tests, etc.[95] When performing and interpreting such test it is vitally important to keep in mind the sensitivity of the dynamic ground parameters to the confining effective stress. Such tests therefore need to be performed under the right stress conditions, e.g. by properly preloading a dynamic plate load test, or the stress dependency needs to be corrected for in the interpretation. An advantage of these in-situ methods is their ability to test the materials at higher dynamic strains, into the elastic non-linear and non-linear range.

It is noted that there are other geotechnical in-situ methods used to obtain specific data such as the pressure meter (ISO 22476, AFNOR XP P94-110-2 and Reference [17]), that is common in some regions. The most common devices do only provide static ground parameters, however there also devices that can determine in-situ cyclic properties of the ground. However, these devices are designed to measure ground properties at high strain levels and are not suited for ground-borne noise and vibration from rail systems.

6.6 Laboratory measurement of dynamic ground parameters

6.6.1 General

In general, laboratory measurements such as the resonant column test are not used to obtain dynamic ground parameters in rail system projects, although they could form a strategy to enhance other investigation methods, and are more utilized in research projects.

Dynamic parameters of ground materials may also be measured in the laboratory on retrieved samples from the site. The main advantage of doing the tests in the laboratory, as opposed to in-situ, is the more controlled conditions offered by the laboratory devices. The major disadvantage is disturbance of the samples and non-representative samples.

Elastic shear and dilatational wave speeds in ground materials may be measured in the laboratory as well as in-situ. The advantage of such measurements in the laboratory is the ability to test the same material under various confining stress conditions, moisture and saturation conditions and degrees of compaction. Measurement of anisotropy is also easier. The major advantage, however, of testing in the laboratory is the ability to control the dynamic (and cyclic strains) and stresses and do measurements where the ground materials have elastic non-linear and purely non-linear behaviour. This is not possible in-situ. Laboratory testing also enables much more reliable and controllable measurement of low-strain material loss factor and makes it possible also to measure the loss factor also in the elastic non-linear and non-linear regime.

The main disadvantage of laboratory measurements is the problem of the tested specimen not necessarily being representative for a ground material in the in-situ conditions of the site. This can either be due to disturbance introduced to the specimen during the sampling and handling process or simply by the fact that one or a few tiny specimens might not represent the overall properties of a large volume of ground material, which controls ground-borne noise and vibration at the actual site.

When considering laboratory versus in-situ testing, one should keep in mind that for ground-borne noise and vibration issues from rail systems, soil behaviour in the low-strain, linear-elastic regime entirely dominates. Only for local issues high up in the track structure or very close to the track, geo-materials can be in the elastic non-linear regime. For well-designed tracks, materials should never be in the really non-linear regime. Focus should therefore be on the low-strain dynamic properties. In soil materials, particularly these low-strain properties are extremely “brittle” and easily distorted by sampling and handling. This favours in-situ measurements over measurements in the laboratory. Strength and high-strain deformation properties as requested for conventional geotechnical and engineering geological design are far more robust and less sensitive to sample disturbance.

Careful sampling can retrieve fairly undisturbed specimens from cohesive materials like clay. A slight disturbance will tend to reduce G_{max} and V_s compared to in-situ conditions. For non-cohesive materials, specimens for laboratory testing usually need to be reconstituted from remoulded material. Elastic properties for such specimens can be far from the in-situ values. Results from reconstituted specimens of material containing silt can particularly deviate from undisturbed in-situ material. Laboratory measurements can be superior for testing fill materials, materials to be compacted, etc. It is, however, important to be aware that coarser materials like ballast, gravel and crushed rock can require far larger samples and testing devices than conventionally used. Laboratory measurements can also be superior to study more fundamental dynamic behaviour of geo-materials, on a research basis.

Laboratory testing of rock materials is usually made on specimens taken from chunks of intact rock from drilled cores. Data from such tests can drastically overestimate wave speed and underestimate the attenuation properties of the in-situ rock mass, since the effect of the joints is missing.

6.6.2 Piezo measurements

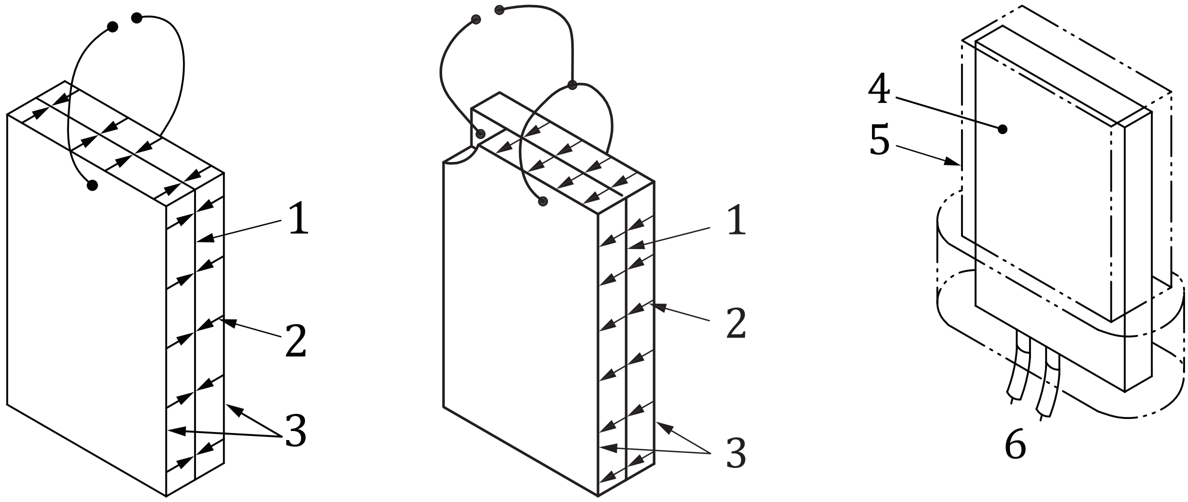
In the laboratory, piezo benders may be used to measure the shear wave speed V_s and thus G_{max} of soil specimens that are under testing within conventional laboratory devices like triaxial, oedometer and direct simple shear (DSS).[27]

Piezo benders are tiny bimorph blade shaped elements and are particularly suited for testing soft soils. They flex when exposed to an electric voltage through their connector leads and produce a voltage when mechanically forced to flex. They can thus be used both as wave transmitters and receivers. In an ordinary setup, the benders are mounted in the top cap and bottom pedestal of the test device and protrude about 5 mm into the specimen as shown in Figure 15. By sending an electrical pulse to the transmitter, a shear wave pulse is generated and propagates through the specimen to the receiver. The travel time is picked from the recorded electrical signals and the shear wave speed calculated from the height of the specimen divided by the travel time, in the same way as done for cross-hole measurements.

The dynamic shear strain of the propagated wave is typically in the range well below 10^{-3} %, and thus clearly within the linear elastic range for the soil material. The shear modulus derived from the wave speed is therefore definitely a measure of G_{max}. Unfortunately, the strain level of the transmitted and received pulse cannot be controlled accurately enough to make the measurements also quantify the loss factor of the material.

A reading by piezo benders is completely non-destructive and performed within a fraction of a second. Since the elements fit so conveniently into ordinary geotechnical testing devices, V_s may be read repeatedly throughout any stage of consolidation and failure testing of the specimen. Piezo benders may also be mounted at the sides of the specimen in perpendicular directions to quantify elastic anisotropy of the ground material at various stress conditions.

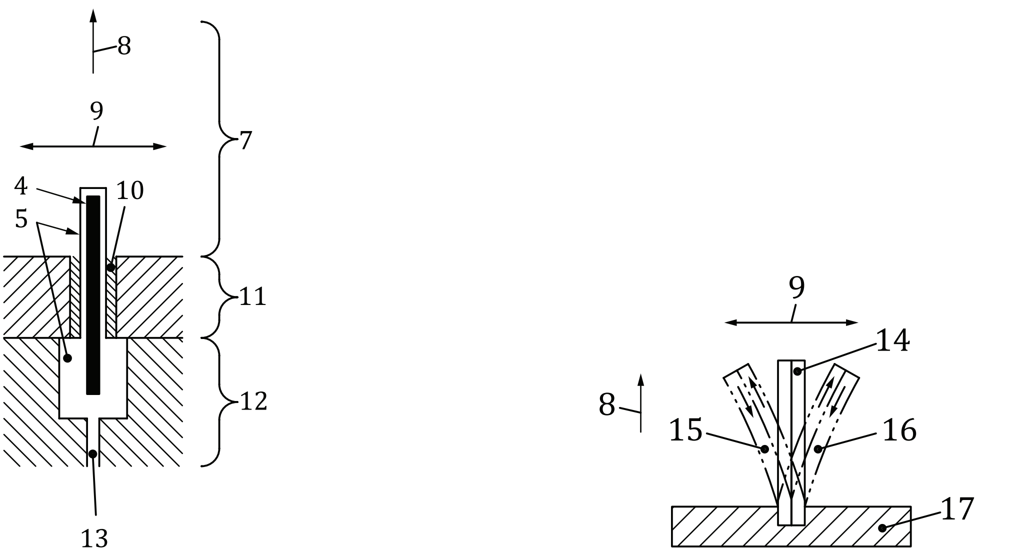
The p- and s-wave speed of rock core specimens may be measured by ultrasonic piezo ceramic elements in the top cap and bottom pedestal of a uniaxial or triaxial testing device, or simply by pushing the crystals against an unconfined specimen. P-wave crystals may also work on soil specimens, particularly if saturated, but s-wave crystals of this type do not work due to the very large impedance contrast between the crystal and the soil. That is the reason why piezo benders are needed for testing soils.



a) Receiver element – series connected piezoceramic elements

b) Transmitter element – parallel connected piezoceramic elements

c) Piezo bender device



d) Piezo bender device arrangement

e) Piezo bender working principle

Key

1 centre electrode
2 direction of polarization
3 surface electrodes
4 piezoceramic bender element
5 epoxy resin casing
6 cable
7 soil specimen
8 direction of shear wave propagation
9 direction of element tip and soil particle movement
10 silicone rubber
11 filter stone
12 pedestal
13 hole for cables
14 zero voltage
15 positive voltage
16 negative voltage
17 bearing plate

Figure 15 — Piezo bender device

6.6.3 Resonant column testing

Whilst the resonant column test is not typically used to evaluate ground dynamic properties in rail projects, it nonetheless has specific advantages and uses that could become part of an overall strategy to supplement the ground investigation.

In a resonant column test the dynamic and cyclic shear modulus (and corresponding shear wave speed) and the material loss factor of soil specimens can be measured in the linear elastic, the non-linear elastic and partly into the non-linear strain range. The dynamic (cyclic) strain is in control and the modulus and damping values versus strain can be deduced. In this context, resonant column testing can be useful to analyse situations where trans-Rayleigh train speeds lead to excessive vibrations and high strains in the track and ground materials.

The basic principle of the resonant column test is based on tuning a cylindrical soil specimen into torsional resonance. From the resonance frequency and the dimensions and the mass of the specimen the dynamic shear modulus can be back figured. The loss factor is determined either from the half-power bandwidth by sweeping the excitation frequency around the resonance, or from the logarithmic decrement of the torsional vibration decay after the excitation has been turned off at resonance. The strain level is controlled through the power of the excitation and the measured response of the specimen.

Most of the available resonant column devices are based on a sort of triaxial cell, where the cylindrical specimen is fixed at the bottom pedestal and sealed in a thin rubber membrane to make the cell pressure impose the radial confining stress. The resonant device is attached to the top of the specimen. There are two commonly used setups.

a) The Stokoe device where a star-shaped plate with four wings is attached to the top of the specimen. The plate is equipped with an accelerometer (and also proximity probes) to monitor its torsional motion and permanent magnets are attached to the tip of each wing. The magnets fit into electrical coils fixed to the frame of the cell. By running AC current through the coils, an oscillatory torque is transferred to the specimen and the frequency and rotation amplitude and thus shear strain can be controlled. In this device, isotropic consolidation of the specimen is the only possibility. The torsional inertia of the star-plate and magnets need to be accounted for in the interpretation of the test results.

b) The Hardin device consists of a torsional oscillator head which is attached to the top of the specimen. The head has a substantial torsional inertial mass and contains torsional springs, an accelerometer and driving magnets and coils. The inertia from the mass acts as counterbalance for the oscillatory torque transferred to the top of the specimen. The oscillator head can transfer axial loads and the specimen can therefore be consolidated anisotropically and can be loaded to failure (if not too strong). Since the oscillator head is a dynamic system by itself, it needs to be included in the dynamic model needed to interpret the tests. The interpretation is therefore more complicated for the Hardin than for the Stokoe device.

Resonant column testing is further described in ASTM D 4015.

Mostly solid cylindrical specimens are used for resonant column testing. The shear strain due to the torsion is therefore not constant over the cross sectional area, but varies from maximum at the perimeter to zero at the centreline. A representative shear strain, usually taken as the strain at 0,8 of the radius of the specimen (see ASTM D 4015), is therefore presented in the results. If more precise control of the dynamic shear strain is requested, a hollow cylindrical specimen may be used in the test. This, however, largely complicates the testing, and is not considered necessary for applications related to ground-borne noise and vibration from rail systems.

Resonant column testing can easily be made at shear strains down to 10^{-4} % and even down towards 10^{-5} % (Hardin device). An excellent coverage of the linear elastic regime is therefore obtained. Towards higher strains 10^{-2} % can easily be reached. For softer soils, it may even be possible to come close to 10^{-1} %, particularly for the Stokoe device which may also be run in a torsional shear mode. A complete coverage of the whole non-linear elastic regime is therefore also obtained. For strains beyond 10^{-2} %, the really non-linear regime can be reached. Here gradual degradation of the soil properties can take place. This necessitates keeping track of the number of load cycles applied.

6.6.4 Cyclic triaxial, DSS and torsional shear testing

Cyclic strains bringing soil materials into the non-linear, degradable regime is usually not an issue for ground-borne noise and vibration from rail systems. If dynamic soil data, like secant shear modulus, damping and generally non-linear stress-strain hysteresis loops, degradation and cyclic strength for this high-strain regime for some reason should be needed, laboratory tests may be run in cyclic triaxial, direct simple shear (DSS) or torsional shear devices. These devices are usually intended for giving soil data for design against ocean wave loading and earthquake loading. The specimen is not brought into any resonating oscillation. The cyclic stresses or strains are rather enforced directly on to the specimen. For this reason, there is practically no limit on how large stresses or strains can be applied. The limitation is on how small strains can be applied in a controlled manner and be measured accurately. Recent development in servo-controlled loading systems and internal-deformation measurements acting directly on the specimen has gradually moved this lower limit downwards. Reliable tests down to 10^{-1} % cause no problem for modern dedicated devices. By particularly fine tuned devices and carefully performed tests it can be possible to reach about 2×10^{-2} % and even 10^{-2} %. In previous days, there was a gap between upper end of the cyclic strain range that could be covered by resonant column tests and the lower end of the strain range covered by cyclic tests (triaxial and direct simple shear). Due to recently improved high-resolution deformation measurements and better test devices, this gap is now near to be closed. More details on cyclic testing of soils are found in Reference [19]. Cyclic testing devices usually operate at typical ocean wave or earthquake frequencies, i.e. in the range 0,1 Hz to 1 Hz. This is low frequency compared to the frequency of typical ground vibration from rail systems. Rate effect corrections can therefore be needed for some ground materials.

7 Strategy for ground parameter determination

7.1 General

How detailed and precise the knowledge about dynamic properties of the ground needed to handle ground-borne noise and vibration issues in an infrastructure or building development project depends on the stage of planning, the size of the project and on the assumed severity of vibration and noise problems. Rough screening methods for parameter estimation can be sufficient at an early conceptual stage while extensive dedicated measurement programs can be needed at the final design of a large and complicated project. The choice of measurement or determination methods for the dynamic parameters should be adapted accordingly.

7.2 Expected severity of ground-borne vibration and noise

Most severe problems associated with low frequency (less than 10 Hz) ground-borne vibration should be expected at sites with soft ground and high water content, short distance between track and buildings and for lines with high axle loads and high speeds. Ground-borne noise will on the other hand particularly be problematic at hard ground and rock sites, and particularly for lines in shallow tunnels and in close proximity to buildings.

7.3 Parameter estimation from available information

At early planning stages of rail system projects, dynamic properties of the ground materials may be estimated with sufficient accuracy from general knowledge on the geology of the site combined with the use of empirical relations to index parameters of the ground. Such index parameters will readily be available from the general geotechnical or engineering geological investigations necessarily done at the site. Available methods are presented in 6.3. Also for projects where ground-borne vibration and noise are not expected to be particularly severe, this simplified approach may give dynamic parameters for the ground, with sufficient accuracy.

7.4 Summary of in-situ measurements versus laboratory measurements

All the available in-situ methods for measuring dynamic properties of the ground are based on some way of measuring the speed of wave propagation and to a limited extent the attenuation properties. To determine the corresponding shear and constrained moduli, information on mass density has to be based on knowledge from the general site investigations. The in-situ methods only reveal data for the linear elastic behaviour. Hardly any reliable information on material damping can be revealed. Some of the methods give averaged data over larger volumes of soil and rock, which is usually beneficial for assessing vibration and noise propagation. Additional methods can give more detailed data at local positions, when needed. Often a combination of these methods gives the optimum combination of overview and resolution.

Laboratory measurements rely on retrieved samples from the site. The major advantage of in-situ measurements over laboratory measurements is that the ground materials are tested in their natural condition. Errors and uncertainties introduced by sample disturbance and non-representative samples are effectively avoided. Laboratory measurements will only reveal data for the precise location of the sample. If the site is inhomogeneous, a large number of samples is needed to obtain the overall picture as obtained by the in-situ methods. Generally, since the in-situ methods in some way measure wave propagation at the site they represent measurements closer to the physics of ground-borne vibration and noise, which is an advantage.

For soils, samples for laboratory measurements are either retrieved undisturbed or are reconstituted in the laboratory from remoulded material. Undisturbed samples are most easily obtained from cohesive soils, e.g. from clays. For undisturbed sampling of non-cohesive soils, e.g. like sands, cumbersome and non-standard “freezing” techniques are needed. The elastic properties of soil materials are usually “brittle”, and even minor sampling disturbance can lead to severely biased data, unless sophisticated sampling techniques are applied. Reconstituted samples usually do not give reliable elastic properties. Sample disturbances usually tend to reduce the elastic moduli relative to the in-situ values.

The major advantage of laboratory testing of soil samples is the ability to also measure properties in the non-linear elastic and non-linear range. In addition, some of the methods do also measure material loss factor, and mass density is always measured. There are possibilities in the laboratory to measure anisotropic properties of the material. During a laboratory testing, index parameters of the soil are determined and various strength and deformation properties are measured. Laboratory tests are therefore well suited to establish site-specific correlations between index parameters and vibration and noise propagation properties.

Often an optimum measurement program consists mainly of in-situ measurements, supplemented with a limited number of laboratory measurements at selected points.

The propagation of vibration and noise through a rock mass is to a larger or lesser extent controlled by the presence and properties of the joints, and not so much by the properties of the intact rock material between the joints. Since laboratory testing of rock usually deals with samples of the intact rock, they do not give information on the wider rock mass properties: Therefore, laboratory testing of rock has a limited value for assessing useful information to evaluate propagation of ground-borne vibration and noise from rail systems.

7.5 Direct measurement of vibration and noise propagation as an alternative to measuring dynamic properties and use of calculation models

The overall objective of measuring dynamic parameters for the ground materials is to use them as input to some sort of calculation model to predict vibration transfer from the track, through the ground and into buildings or structures. An alternative approach to be considered is to measure the vibration or noise transfer directly. One may either do measurements at the actual site using some sort of vibration exciter in an idealized setup to simulate the coming rail system, or one may find a site that is similar enough, where a rail transportation system is already in operation. Vibration transfer into future buildings may be estimated in the same way at equivalent sites.

The direct measurement approach is particularly beneficial at sites where the ground conditions are heterogeneous and hard to map like in urban areas.

Most fruitful are direct measurements combined with measurement of dynamic parameters for the ground and calculation models. Ground parameters and calculations are means to “bridge” measurement results from an idealized setup or an equivalent site over to the real case and thus fine-tune or calibrate the calculation model and input parameter determination.

It is noteworthy that the rail source itself can also be used in propagation studies, which compare vibration at specified distances, which provide information on exponents used in overall ground losses, though would not derive wave speeds or specific material loss parameters.

Annex A
(informative)

Abbreviations used in this part of ISO 14837

1D	one-dimensional;
2D	two-dimensional;
AC	alternating current;
CPT	cone penetration test;
CSW	continuous surface waves;
DSS	direct simple shear (device);
ERT	earth resistivity tomography;
LVDT	linear variable differential transformer;
MASW	multi-channel analysis of surface waves;
OCR	over-consolidation ratio;
S-CPT	seismic CPT;
SASW	spectral analysis of surface waves;
SDOF	single degree of freedom;
SPT	standard penetration test;
TFR	time-frequency representation;
VSP	vertical seismic profiler.

Bibliography

[1] ISO 8041, *Human response to vibration — Measuring instrumentation*

[2] ISO 14688 (all parts), *Geotechnical investigation and testing — Identification and classification of soil*

[3] ISO 22476 (all parts), *Geotechnical investigation and testing — Field testing*

[4] ISO 26824, *Particle characterization of particulate systems — Vocabulary*

[5] EN 1998-5, *Eurocode 8: Design of structures for earthquake resistance — Part 5: Foundations, retaining structures and geotechnical aspects*

[6] AFNOR NF P94-110-1, *Sols: Reconnaissance et essais — Essai pressiométrique Ménard — Partie 1: Essai sans cycle*

[7] AFNOR XP P94-110-2, *Sols: Reconnaissance et essais — Essai pressiométrique Ménard — Partie 2: Essai avec cycle*

[8] ASTM D 3441, *Standard test method for mechanical cone penetration tests of soil*

[9] ASTM D 4015, *Standard test methods for modulus and damping of soils by resonant-column method*

[10] ASTM D 4428/D 4428M, *Standard test methods for crosshole seismic testing*

[11] ASTM D 5778, *Standard test method for electronic friction cone and piezocone penetration testing of soils*

[12] ASTM D 6429, *Standard guide for selecting surface geophysical methods*

[13] ASTM D 7400, *Standard test methods for downhole seismic testing*

[14] DIN 45669-2, *Measurement of vibration immission — Part 2: Measuring method*

[15] DIN 45672-1, *Schwingungsmessungen in der Umgebung von Schienenverkehrswegen — Teil 1: Messverfahren*

[16] Lamb T.W., & Whitman R.V. Soil mechanics. SI version. Wiley, New York, 1979

[17] Clayton C.R.I, Matthews M.C., Simons N.E. Site investigation. Blackwell Science, Oxford, 1995

[18] Sheriff R.E., & Geldart L.P. Exploration seismology. Cambridge University Press, 1995

[19] Bowles J.E. Engineering properties of soils and their measurement. McGraw-Hill Book Company, 1978

[20] Kramer S.L. Geotechnical earthquake engineering. Prentice Hall, New Jersey, 1996

[21] Deutsche Gesellschaft für Geotechnik. Empfehlungen des Arbeitskreises 1.4 Baugrunddynamik, 2001

[22] Robertson P.K. Interpretation of cone penetration tests – A unified approach. Canadian Geotechnical Journal, 2009, **46** pp. 1337–1355

[23] Silver M.L. , & Tiedemann D. eds. Dynamic geotechnical testing. ASTM publication STP 654, 1977

[24] Ebelhar R.J. , Drnevich V.P. , Kutter B.L. eds. Dynamic geotechnical testing II. ASTM publication STP 1213, 1994

[25] Butcher A.P., Campanella R.G., Kaynia A.M., Massarch K.R. Seismic cone down-hole procedure to measure shear wave velocity – A guideline prepared by ISSMGE TC10: Geophysical

Testing in Geotechnical Engineering. Proc. XVIth International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, Japan, 2005

[26] Andersen K.H., Kleven A., Heien D. Cyclic Soil data for design of gravity structures. J. Geotech. Eng., 1988, **114** (5)

[27] Dyvik R., & Madshus C. Lab measurements of $G_{\max}$ using bender elements. Advances in the art of testing soils under cyclic conditions. Proceedings of a session in conjunction with the ASCE Convention in Detroit, Michigan 1985, American Society of Civil Engineers, pp. 186–196

[28] Madshus C. Soil non-linearity and its effect on the dynamic behaviour of offshore platform foundations. PhD. Department of Geology, University of Oslo, 1997

[29] Vucetic M. Cyclic threshold shear strains in soils. J. Geotech. Eng., 1994, **120** (12), pp. 2208–2228

[30] Vucetic M., & Dobry R. Effect of soil plasticity on cyclic response. J. Geotech. Eng., 1991, **117** (1), pp. 89–107

[31] Vucetic M., & Dobry R. Degradation of marine clays under cyclic loading. J. Geotech. Eng., 1988, **114** (2) pp. 133–149

[32] Larsson R., & Mulabdic M. Shear moduli in Scandinavian clays. Measurement of initial shear modulus with seismic cones. Empirical correlations for the initial shear modulus in clay. Swedish geotechnical institute, Report No 40, Linköping, 1991

[33] Ohta T., & Goto N. Estimation of s-wave velocity in terms of characteristic indices of soils. Butsuri Tanko, 1976, **29** (4) pp. 34−41

[34] Seed H.B., Wong R.T., Idriss I.M., Tokimatsu K. Moduli and damping factors for dynamic analyses of cohesionless soils. J. Geotech. Eng., 1986, **112** (GT11) pp. 1016–1032

[35] Imai T., & Tonouchi K. Correlation of N-values and s-wave velocity and shear modulus. Proc. 2nd European Symposium on Penetration Testing, Amsterdam, 1982, pp. 57–72

[36] Lunne T., Robertson P. K., Powell J. J. M. Cone penetration testing in geotechnical practice. London, Blackie Academic and Professional, XXIII, 1997

[37] Semblat J.F., & Pecker A. *Waves, traffic, shocks, construction works*. IUSS Press, Pavia, Italy, 2009

[38] Barton N. Rock quality, seismic velocity, attenuation and anisotropy. Taylor and Francis/Balkema, London, 2007

[39] Mavko G., Mukerji T., Dvorkin J. The rock physics handbook. Cambridge University Press, 1998

[40] Biot M.A. Theory of propagation of elastic waves in a fluid-saturated porous solid, I, Low-frequency range. J. Acoust. Soc. Am., 1956, **28** pp. 168–178

[41] Stoll R.D., & Kan T.K. Reflection of acoustic waves at a water-sediment interface. J. Acoust. Soc. Am., 1981, **77** pp. 149–156

[42] Kaynia A.M., Løvholt F., Madshus C. Effects of a multi-layered poro-elastic ground on attenuation of acoustic waves and ground vibration. J. Sound Vibrat., 2011, **330** (7) pp. 1403–1418

[43] Sjøgren B. Shallow refraction seismics. Chapman & Hall, London, 1984

[44] Darendeli M.B., Stokoe K.H., Gilbert R.H. Development of a new family of normalized modulus reduction and material damping curves. University of Texas, 2001

[45] Darendeli M.B. Development of a new family of normalized modulus reduction and material damping curves. Ph.D. University of Texas, 2001

[46] Stokoe K.H., Lee S.H.H., Knox D.P. Shear moduli measurements under true triaxial stresses. Advances in the art of testing soils under cyclic conditions. Proc. ASCE Convention in Detroit, USA, 1985

[47] Menq F.Y. Dynamic properties of sandy and gravelly soils. Ph.D. University of Texas, 2003

[48] MAYNE P.W., & RIX G.J. $G_{\text{max}} - q_{\text{c}}$ relationships for clay. *Geotechnical Testing Journal*, ASTM 16, No 1, 1993, pp. 54–60

[49] Aki K., & Richards P.G. Quantitative seismology. University Science Books, 2002

[50] Caiti A., Akal T., Stoll R.D. Estimation of shear wave velocity in shallow marine sediments. IEEE J. Oceanic Eng., 1994, **19** pp. 58–72

[51] Constable S.C., Parker R.L., Constable C.G. Occam's Inversion: a practical algorithm for generating smooth models from electromagnetic sounding data. Geophysics, 1987, **52** pp. 289–300

[52] Dziewonski A., Bloch S., Landisman M. A technique for the analysis of transient seismic signals. Bull. Seismol. Soc. Am., 1969, **55** pp. 427–444

[53] Dziewonski A.M., & Hales A.L. Numerical analysis of dispersive seismic waves. In: Methods in Computational Physics, (Bolt B.A. ed.). Academic Press, 1972, pp. 271–95.

[54] Forbriger T. Inversion of shallow-seismic wavefields: I. Wavefield transformation. Geophys. J. Int., 2003, **153** pp. 719–734

[55] Foti S. Multi-station methods for geotechnical characterization using surface waves, PhD, Torino, Italy, 2000

[56] Foti S. Using transfer function for estimating dissipative properties of soils from surface-wave data, *Near Surface Geophysics.* 2004, **2**, pp. 231–240

[57] Herrmann R.B. Some aspects of band-pass filtering of surface waves. Bull. Seismol. Soc. Am., 1973, **63** pp. 663–671

[58] Hisada Y. An efficient method for computing Green's functions for a layered half-space with sources and receivers at close depths. Bull. Seismol. Soc. Am., 1994, **84** pp. 1456–1472

[59] Holschneider M., Diallo M.S., Kulesh M., Ohrnberger M., Lück E., Scherbaum F. Characterization of dispersive surface waves using continuous wavelet transforms. Geophys. J. Int., 2005, **163** pp. 463–478

[60] Auersch L. Technically induced surface wave fields. Part I: Measured attenuation and theoretical amplitude-distance laws. Part II: Measured and calculated admittance spectra. Bull. Seismol. Soc. Am., Vol. 100, No 4, 2010, pp. 1528–1539 and 1540–1550

[61] Kritski A., Vincent A.P., Yuen D.A., Carlsen T. Adaptive wavelets for analysing dispersive seismic waves. Geophysics, 2007, Vol. 72

[62] Lai C.G. Simultaneous inversion of Rayleigh phase velocity and attenuation for near-surface site characterization. Ph.D. Georgia Institute of Technology, 1998

[63] Luo Y., Xia J., Miller R., Xu Y., Liu J., Liu Q. Rayleigh-wave mode separation by high-resolution linear Radon transform. Geophys. J. Int., 2009, **179** pp. 254–264

[64] Luo Y., Xu Y., Liu Q., Xia J. Rayleigh-wave dispersive energy imaging and mode separating by high-resolution linear Radon transform. Leading Edge (Tulsa Okla.), 2008, **27** pp. 1536–1542

[65] McMechan G.A., & Yedlin M.J. Analysis of dispersive wave by wave field transformation. Geophysics, 1981, **46** pp. 869–874

[66] Mosegaard K., & Sambridge M. Monte Carlo analysis of inverse problems. Inverse Probl., 2002, **18** pp. 29–54

[67] Park C.B., Miller R.D., Xia J. Multichannel analysis of surface waves. Geophysics, 1999, **64** pp. 800–808

[68] Park C.B., Miller R.D., Xia J. Multimodal analysis of high-frequency surface waves. In Symposium on the Application of Geophysics to Engineering and Environmental Problems, 1999, pp. 115–121, Oakland, USA

[69] Park C.B., Miller R.D., Xia J., Ivanon J. Multichannel analysis of surface waves (MASW) – active and passive methods. Leading Edge (Tulsa Okla.), 2007, **26** pp. 60–64

[70] Polom U., Hansen L., Sauvin G., L'Heureux J.-S., Lecomte I., Krawczyk C. High-resolution SH-wave reflection seismics for characterization of onshore ground conditions in the Trondheim harbour, central Norway. In Advances in Near-Surface Seismology and Ground-Penetrating Radar, eds. Miller, R. D., Bradford, J. D. and Holliger, K. Society of Exploration Geophysicists (SEG), 2010, pp. 297–312

[71] Pugin A.J.M., Pullan S.E., Hunter J.A. Multi-component high-resolution seismic reflection profiling. Leading Edge (Tulsa Okla.), 2009, **28** pp. 1248–1261

[72] Rix G.J. Experimental study of factors affecting the spectral-analysis-of-surface-waves method. Ph.D. University of Texas, 1988

[73] Rix G.J., & Stokoe K.H. Correlation of initial tangent modulus and cone penetration resistance. Calibration Chamber Testing. International Symposium on Calibration Chamber Testing, A.B. Huang, ed. Elsevier Publishing, New York, 1991, pp. 351–362

[74] Ryden N., & Park C.B. Surface waves in inversely dispersive media. *Near Surface Geophysics.*, 2004, **2**, pp. 187–197

[75] Sambridge M., & Mosegaard K. Monte Carlo Methods in Geophysical Inverse Problems. Rev. Geophys., 2002, **40** p. 1009

[76] Sauvin G. Shear Wave Seismic for Geohazards – A Case Study from Trondheim Harbour. International Centre for Geohazards. ICG, Oslo, 2009

[77] Socco L.V., & Boiero D. Improved Monte Carlo inversion of surface wave data. Geophys. Prospect, 2008, **56** pp. 357–371

[78] Socco L.V., & Strobbia C. Surface-wave method for near-surface characterization: A tutorial. *Near Surface Geophysics.*, 2004, **2**, pp. 165–185

[79] Socco V.L., Maraschini M., Boiero D., Vanneste M., Madshus C., Westerdahl H. On the use of NGI's prototype seabed-coupled shear wave vibrator for shallow soil characterization – Part II: Joint inversion of multi-modal Love and Scholte surface waves. Geophys. J. Int., 2011, **185** (1) pp. 237–252

[80] Vanneste M., Madshus C., Socco L.V., Maraschini M., Sparrevik P.M., Westerdahl H. On the use of NGI's prototype seabed-coupled shear wave vibrator for shallow soil characterization – Part I: Acquisition and processing of surface waves. Geophys. J. Int., 2011, **185** (1) pp. 221–236

[81] Stokoe K.H. , Lee N.J. , Rits M.P. Repeated measurements of in situ soil stiffness with permanently embedded geophones. In: Ebelhar R.J., Drnevich V.P., Kutter B.L., eds. Dynamic Geotechnical Testing. ASTM, 1994

[82] Stokoe K.H., Wright S.G., Roesset J.M., Gauer R., Sedighi-Manesh M. In situ measurement of stiffness profiles in ocean bottom materials using the SASW method. In Offshore Technology Conference, Texas, USA, 1990

[83] Wathelet M., Jongmans D., Ohrnberger M. Surface-wave inversion using a direct search algorithm and its application to ambient vibration measurements. *Near Surface Geophysics*, 2004, **2**, pp. 211–221

[84] Xia J., Xu Y., Chen C., Kaufmann R.D., Luo Y. Simple equations guide high-frequency surface-wave investigation techniques. Soil. Dyn. Earthquake Eng., 2006, **26** pp. 395–403

[85] Xia J., Xu Y., Miller R.D. Generating an image of dispersive energy by frequency decomposition and slant stacking. Pure Appl. Geophys., 2007, **164** pp. 941–956

[86] Yamanaka H., & Ishida H. Application of genetic algorithm to an inversion of surface wave dispersion data. Bull. Seismol. Soc. Am., 1996, **86** pp. 436–444

[87] Yilmaz O. Seismic data analysis: Processing, inversion and interpretation of seismic data. Society of Exploration Geophysicists, Tulsa, **10**, Second Edition, 2001

[88] Harris C.M. Shock and vibration handbook. McGraw-Hill, 1995

[89] Cremer L., Heckl M., Ungar E.E. Structure-borne sound. Springer, Berlin, 1988

[90] Blake F.G. Spherical wave propagation in solid media. J. Acoust. Soc. Am., 1952, **24** (2)

[91] Kolsky H. Stress waves in solids. Dover publications, New York, 1964

[92] White J.E. Underground sound – Application of seismic waves. Elsevier, 1983

[93] National Cooperative Highway Research Program Synthesis 368. Cone penetration testing – A synthesis of highway practice. Transportation Research Board, Washington, D.C., 2007

[94] Telford W.M., Geldart L.P., Sheriff R.E. Applied geophysics. Cambridge University Press, Second Edition, 1990

[95] Bodare A. Dynamic screwplate for determination of soil modulus in situ. Ph.D. Institute of Technology, Uppsala University, 1983

[96] Swiss Geophysical Commission. Quality guideline for geophysical methods. www.gr.sgpk.ethz.ch

[97] Stokoe K.H., & Santamarina J.C Seismic-wave-based testing in geotechnical engineering. GeoEng2000, An international conference on geotechnical and geological engineering, Melbourne, Australia, Vol. 1, 2000, pp. 1490–1536

[98] Andersen K.H., Lunne T., Kvalstad T.J., Forsberg C.F. Deep water geotechnical engineering. Proc. XXIV National Conf. Of the Mexican Soc. Of Soil Mechanics, Aguacalientes, 2008

[99] Andersen K.H. Cyclic clay data for design of structures subjected to wave loading. International Conference on Cyclic Behaviour of Soils and Liquifaction Phenomena, Bochum, Germany, 2004

[100] Design standards for railway structures and commentary (Foundation). Railway Technical Research Institute, Japan

[101] Design specifications for highway bridges — Part V: Seismic design. Japan Road Association

[102] Madshus C., & Kaynia A.M. High-speed trains on soft ground: track-embankment-soil response and vibration generation. Noise and vibration from high-speed trains, V.V. Krylov, editor. Th. Telford, 2001, pp. 315–346

[103] Schnaid F. In-situ testing in geomechanics. Taylor and Francis, 2009

[104] Robertson P.K. Interpretation of in-situ tests – Some insights. Mitchell Lecture – ISC'4 Brazil, 2012

[105] Test procedures for determination of dynamic soil characteristics. Deliverable D1.1 to EU 7th Framework program project RIVAS – Railway induced vibration abatement solutions collaborative project, UIC, 2011, www.rivas.eu

[106] Schevenels M., Degrande G., Lombaert G. The influence of the depth of the ground water table on free field traffic-induced vibrations. Int. J. Numer. Anal. Methods Geomech., 2004, **28** pp. 395–419

[107] Schevenels M., Lombaert G., Degrande G., Francois S. A probalistic assessment of the resolution in the SASW test and its impact on the prediction of ground vibrations. Geophys. J. Int., 2008, **172** pp. 262–275

[108] Thornely-Taylor R. Numerical modelling of groundborne noise and vibration from underground railways: Geotechnical considerations. Proc. 12th International Congress on Sound and Vibration, ICSV 12, Lisbon, 2005

This page deliberately left blank

British Standards Institution (BSI)

BSI is the national body responsible for preparing British Standards and other standards-related publications, information and services.

BSI is incorporated by Royal Charter. British Standards and other standardization products are published by BSI Standards Limited.

About us

We bring together business, industry, government, consumers, innovators and others to shape their combined experience and expertise into standards -based solutions.

The knowledge embodied in our standards has been carefully assembled in a dependable format and refined through our open consultation process. Organizations of all sizes and across all sectors choose standards to help them achieve their goals.

Information on standards

We can provide you with the knowledge that your organization needs to succeed. Find out more about British Standards by visiting our website at bsigroup.com/standards or contacting our Customer Services team or Knowledge Centre.

Buying standards

You can buy and download PDF versions of BSI publications, including British and adopted European and international standards, through our website at bsigroup.com/shop, where hard copies can also be purchased.

If you need international and foreign standards from other Standards Development Organizations, hard copies can be ordered from our Customer Services team.

Subscriptions

Our range of subscription services are designed to make using standards easier for you. For further information on our subscription products go to bsigroup.com/subscriptions.

With **British Standards Online (BSOL)** you'll have instant access to over 55,000 British and adopted European and international standards from your desktop. It's available 24/7 and is refreshed daily so you'll always be up to date.

You can keep in touch with standards developments and receive substantial discounts on the purchase price of standards, both in single copy and subscription format, by becoming a **BSI Subscribing Member**.

PLUS is an updating service exclusive to BSI Subscribing Members. You will automatically receive the latest hard copy of your standards when they're revised or replaced.

To find out more about becoming a BSI Subscribing Member and the benefits of membership, please visit bsigroup.com/shop.

With a **Multi-User Network Licence (MUNL)** you are able to host standards publications on your intranet. Licences can cover as few or as many users as you wish. With updates supplied as soon as they're available, you can be sure your documentation is current. For further information, email bsmusales@bsigroup.com.

Revisions

Our British Standards and other publications are updated by amendment or revision.

We continually improve the quality of our products and services to benefit your business. If you find an inaccuracy or ambiguity within a British Standard or other BSI publication please inform the Knowledge Centre.

Copyright

Useful Contacts:

Customer Services
Tel: +44 845 086 9001
Email (orders): orders@bsigroup.com
Email (enquiries): cservices@bsigroup.com

Subscriptions
Tel: +44 845 086 9001
Email: subscriptions@bsigroup.com

Knowledge Centre
Tel: +44 20 8996 7004
Email: knowledgecentre@bsigroup.com

Copyright & Licensing
Tel: +44 20 8996 7070
Email: copyright@bsigroup.com

BSI Group Headquarters

389 Chiswick High Road London W4 4AL UK