06 Apr 1995, 8:00 am - 9:00 am

Wave Propagation in the Ground and Isolation Measures

W. A. Haupt
Landesgewerbeanstalt Bayern, Nuremberg, Germany

Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 2–7, 1995, Volume II, St. Louis, Missouri

Wave Propagation in the Ground and Isolation Measures

Paper No. SOA11

(State of the Art Paper)

W.A. Haupt
Landesgewerbeanstalt Bayern, Nuremberg, Germany

SYNOPSIS The first part of this report deals with wave propagation in the ground. Special attention is given to the radiation of waves from a source and on the effect of layering or continuous variation of the shear modulus in the soil on travelling waves. In the second part, vibration isolation measures are described, such as open or infilled trenches and rows of bore holes or piles. The results of different theoretical and experimental investigations are compared. Finally, recently developed isolation measures are presented.

1 INTRODUCTION

The propagation of waves in soil is a fundamental problem in earthquake science and soil dynamics. It is this physical phenomenon, by which the dynamic energy is transferred through the soil from the source to the place of concern. Of substantial significance for the transportation of energy is not only the type of waves generated but also the dynamic properties of the soil, the degree of inhomogeneity and the general boundary conditions. The investigation of the effects of these parameters on the propagation of waves is therefore an important problem in soil dynamics. Furthermore, by measuring wave propagation at the surface or in the subsurface soil, the engineer may be able to detect the dynamic behaviour of the ground, which is necessary e.g. for the earthquake-resistant design of a structure. Special questions arise with topographic irregularities or inhomogeneities in the ground. Among other things, they may have an isolating effect on travelling waves and can therefore be used as measures to protect sensitive objects against soil vibrations.

2 BODY WAVES

2.1 FUNDAMENTAL RELATIONSHIPS

The derivation of the fundamental differential equations of wave propagation and their solutions may be found in quite a number of good textbooks, e.g. Love (1944), Ewing et al. (1957), Barkan (1962), Kolsky (1963), Richart et al. (1970), Prakash (1981), Haupt (1986), Studer/Ziegler (1986). Therefore, only some basic information on body waves is presented here. Some features which are not readily found in the textbooks are considered in more detail.

The basic equation of a plane, harmonic (sinusoidal) wave propagating in x-direction is given by:

$$u(x,t) = u_o\, e^{\,i(\omega t - kx)} \qquad (1)$$

where u_0 = amplitude, ω = circular frequency, k = wave number, i = -1 . Important relations are:

$$\omega = 2\pi f; \qquad k = \omega / V; \qquad V = f \cdot L \qquad (2)$$

with f = frequency, V = wave velocity and L = wave length. Equation (1) is illustrated in Fig. 1 in the complex plane. The actual displacement u (x, t) is the real part of the vector and the phase angle φ is given by the position of the vector. This angle, as defined in Fig. 1, increases with x and decreases with t. The slope of the

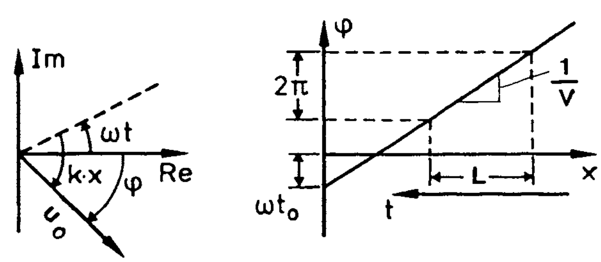


Fig. 1: Phase angle in the complex plane

line is proportional to 1/V. One should always bear in mind the ambivalent character of the wave:

- at t = constant the wave is a harmonic function of the spatial variable x;
- at x = constant the wave appears as a harmonic vibration dependent on the time t at a given point.

In an unbounded body of a linear elastic material there are two types of waves (Fig. 2):

- Compression wave (P-wave, longitudinal or dilatational wave). The displacements of a point due to the passing wave are parallel to the direction of propagation (longitudinal). This wave is associated with volumetric strain and shear deformation (subscript P).
- Shear wave (S-wave, transversal or distortional wave). In this case the displacements of a point are perpendicular (transverse) to the direction of propagation of the wave. This wave is associated purely with a shear deformation. No dilatation takes place (subscript S).

The velocities of propagation of these waves are:

$$V_S = \sqrt{\frac{G}{\rho}}; \quad V_P = \sqrt{\frac{G}{\rho}} \cdot \sqrt{\frac{2(1-\nu)}{1-2\nu}} \tag{3}$$

where G = shear modulus, ρ = density and ν = Poisson's ratio. V_P is always greater than V_S by a factor ranging from 1.4 (ν = 0) to ∞ (ν = 0.5).

2.2 DAMPING

With waves propagating through the ground, a small amount of the transferred elastic energy is always transformed to other kinds of energy (e.g. heat). The dissipation of wave energy is due to what is called the material damping of the soil. If we assume a complex shear modulus $G' = G(1 + i\,\psi/2\pi)$, with ψ being the specific damping capacity, the equation (1) becomes

$$u(x,t) = u_0\, e^{i(\omega t - kx)} \cdot e^{-\alpha x} \tag{4}$$

Hence, the amplitude of the harmonic wave is attenuated in the form of an exponential function (Bornitz, 1931). The coefficient of attenuation is calculated from $\alpha = (\psi \cdot f)/(2 \cdot V)$ (Kolsky, 1963). If the wave velocity V is constant, then α is directly proportional to the frequency. This explains the common experience that high frequency vibrations in soil decay much faster with distance than low frequency ones. Therefore, with a pulse propagating through the ground, the dominant frequency is shifted from the high values to the low values with travel time.

The material damping in soil is usually considered to be of the friction type, which implies ψ having a constant value. However, very often, especially in theoretical investigations, a linear visco-elastic material behaviour is assumed (Voigt-Kelvin-body), which is not far from reality for very small strain levels. In such a material, the damping capacity and consequently the complex modulus increase with frequency. A material with frequency dependent wave velocity is called dispersive. There are two kinds of wave velocity in this medium:

- the phase velocity is the velocity of propagation of the harmonic, steady-state wave at a given frequency;
- the group velocity is that velocity at which the transport of the energy of a pulse, i.e. of a transient wave containing some range of frequency, takes place.

With the usually small amount of material damping of soil, there is only little influence on the wave velocity, at maximum about 2% (Haupt, 1978a). The commonly encountered frequency dependency of the surface wave velocity is not due to material but to geometrical dispersion.

2.3 WAVE VELOCITY IN SATURATED SOIL

Ground water in the soil is a common phenomenon. If the pores of the soil are fully saturated with water, there are three different waves in this two-phase system (Biot, 1956):

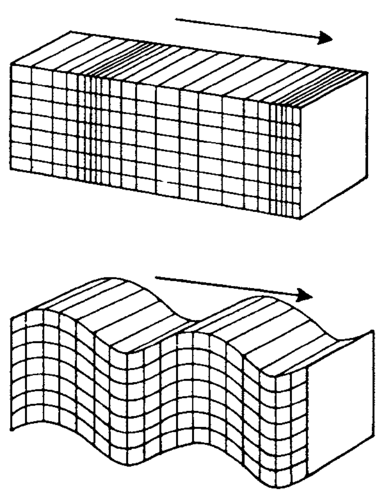


Fig. 2: Deformation of a body due to P- and S-wave

- a compressional wave transferred by the pore water (first kind);
- a compressional wave (second kind) and
- a shear wave, both propagating through the elastic grain skeleton of the soil.

As long as the frequency is smaller than a characteristic frequency f_c defined by Ishihara (1970), the water-filled soil can be considered approximately as a medium to which the theory of elasticity applies. However, in this case the mass of the pore water has to be added to the soil mass for the density of the mixture. On the other hand, the bulk modulus of the water is increased due to the much greater modulus of the soil grains which make up part of the mixture. Both relations from Wood (1930) are dependent on the void ratio e. From this, it follows that in a dense soil, the velocity of the compressional wave (first kind) in the mixture is greater than that in pure water, which is about 1440 m/s (at 8 °C). If, however, the void ratio becomes greater than about unity, the wave velocity in the mixture decreases below this reference velocity, as may be seen from Fig. 3. Hardin (1961) has shown that the dilatational wave velocity (first kind) of the mixture is almost independent of the state of external stress. The velocity of the shear wave in the soil structure is slightly reduced by about 5% to 15%, due to the increase of density by the pore water.

The characteristic frequency f_c depends on the porosity of the soil (n) and on the coefficient of permeability (k) and its value is in the range of kHz. At higher frequencies a decoupling of water and grain skeleton takes place, thus increasing the dilatational wave velocity in the mixture (Allen et al., 1980; Wu/Shen, 1991). The compressional wave velocity in the pore water is drastically reduced by even very small amounts of air bubbles.

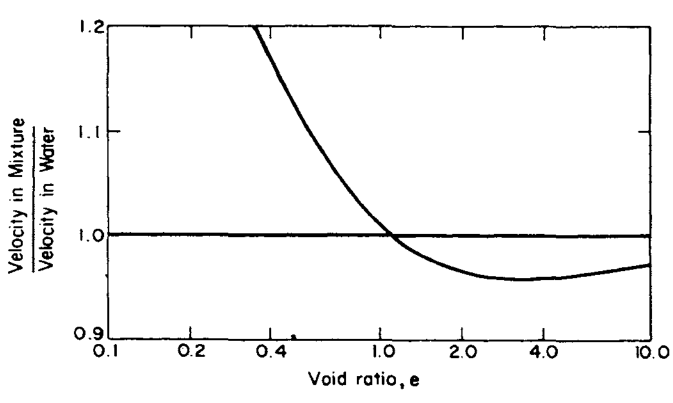


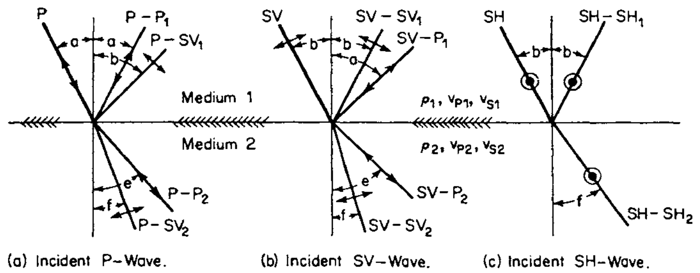
Fig. 3: Compression wave velocity in soil-water-mixture (Richart et al., 1970)

At a degree of saturation of 99.9% the velocity is decreased to about ¼ of that in a fully saturated soil (Richart et al., 1970; Bardet/Sayed, 1993). Allen et al. (1980) have proven experimentally the dramatic decrease of wave velocity with only a few air bubbles. A three-phase model of the soil (grains/water/air) has been developed by Vardoulakis/Beskos (1986). Wu/Shen (1991) have proposed a procedure to determine Poisson's ratio, porosity and density of the soil based on the measurement of the dilatational wave velocity of the pore water. The theory of Biot and successors has been implemented into numerical calculation models which make possible the investigation of surface waves propagating in submerged soil layers (Hirai, 1992; Nogami/Kazama, 1992) or of the influence of physical soil parameters like permeability or degree of saturation on the vibration of structures (Cramer/Wunderlich).

2.4 BODY WAVES AT INTERFACES

At an interface between two elastic bodies the waves are partly reflected as well as refracted. Due to the conditions of continuity at the interface, not only is the type of the incident wave concerned, but the other wave type is also generated, as may be seen in Fig. 4. The angles of reflection and refraction are determined by Snell's law. The amplitudes of the concerned waves depend on the angle of incidence in the form of complicated functions (Zoeppritz, 1919). McCamy et al. (1962) have shown that the essential parameter of influence is the ratio of the wave velocities, whereas ϱ_1/ϱ_2 and ν_1/ν_2 are less important. A number of informative examples can be found in Ewing et al. (1957) and Richart et al. (1970). If the incident angle is 0°, i.e. perpendicular incidence, only the wave of the same type is reflected and transmitted. In a bar, each variation of the wave impedance, whether due to change in cross-sectional area or material stiffness, gives rise to a partial reflection of the wave travelling along the bar. At an open end full reflection occurs. This phenomenon is utilized in the seismic integrity measurement of piles.

From Fig. 4, it follows that in the case of $V_2 > V_1$, the angle of the refracted wave is greater than the angle of the incident wave. If the angle of refraction becomes 90°, the angle of incidence is called critical angle. Beyond this angle no propagation of energy into the second medium takes place. The critical angle plays an essential role in the seismic refraction or reflection survey in a layered ground. Under certain conditions it is possible that a wave propagates along the interface between two elastic materials. The velocity of propagation of the so-called



$$\frac{\sin a}{v_{P1}} = \frac{\sin b}{v_{S1}} = \frac{\sin e}{v_{P2}} = \frac{\sin f}{v_{S2}}$$

Fig. 4: Elastic waves at an interface (Richart et al., 1970)

Stoneley wave (Stoneley, 1924) is in the order of the S-wave velocity of the faster medium.

2.5 EFFECT OF STRESS-ANISOTROPY

Based on a large number of research results and practical experience, the shear wave velocity is usually considered to depend on the confining pressure in the soil as a power function. The state of stress is characterized by the isotropic stress and the exponent is in the order of 0.25. Hardin/Black (1966) have shown that instead of the isotropic stress the mean principal stress $\bar{\sigma}_0 = (\sigma_1 + \sigma_2 + \sigma_3)/3$ may be used as a good approximation. This relationship is widely accepted today.

However, by more detailed investigations Roesler (1979), Stokoe et al. (1985) and Fei/Richart (1991) have found the respective principal stresses to have different influence on the wave velocity. They measured the velocity primarily of polarized shear waves by seismic methods in sand bodies where the principal stresses could be varied indepently of each other. If

σ_a = principal stress in direction of propagation
σ_p = principal stress in direction of particle motion
σ_s = principal stress perpendicular to the plane of σ_a - σ_p

then V_S depends only on σ_a and σ_p as a power function with the exponents na and np being about 0.1 to 0.15 respectively. The principal stress component σ_s has no influence on V_S. Roesler has proposed a relationship

$$V_S \sim (\sigma_a)^{na} \cdot (\sigma_p)^{np} \cdot (\sigma_s)^{ns}, \tag{5}$$

where ns = 0, if the wave propagates along the axes of the principal stresses. This relationship was confirmed by the test results by Stokoe et al. (1985). They also re-evaluated earlier test results published by other researchers and found similar values for na and np, especially ns being zero.

From resonant-column test results on sand under anisotropic stress conditions, Yu/Richart (1984) concluded that the expression for the mean principle stress should be modified to $\bar{\sigma}_{02} = (\sigma_a + \sigma_p)/2$. Equally, Fei/Richart (1991) found that their test results, although consistent with those obtained by Roesler and by Stokoe et al., could well be described by applying the 0.25 power to $\bar{\sigma}_{02}$. Moreover, if the waves were travelling at an angle to the principal stress direction, in eq. (5) the sum of the exponents was too high. Therefore, for this kind of wave propagation, the authors propose calculating the shear wave velocity as usual by using $\bar{\sigma}_0$ and the exponent equal to 0.25.

Experimental investigations dealing mainly with the compression wave velocity under anisotropic conditions were carried out by Lee (1991). Measurement directions were not only the principal stress axes but also directions under five different angles between these axes. Firstly, by seismic methods, a structural anisotropy of the sand body with the horizontal plane being the plane of isotropy was detected. With the wave propagation along the axes of principal stress, the shear wave velocity again turned out to depend only on σ_a and σ_p as shown in eq. (5) but not on σ_s. The compressional wave velocity, however, for these conditions was found to depend on σ_a alone. Both other principal stresses showed no influence. This can be explained by the fact that at this wave type both the direction of propagation and that of the particle motion are identical. Stokoe et al. (1991) have shown that the stress-induced anisotropy of the material can be described by a cross-anisotropic model by Love (1944). The five independent elastic constants which are required for this model could be determined by the seismic wave velocity measurements.

3 WAVE PROPAGATION AT THE SOIL SURFACE

3.1 RAYLEIGH WAVE

The so-called Rayleigh wave is a special type of wave which can only exist at the stress-free surface of the half-space. It was first mathematically described by Lord Rayleigh (1885). Since the derivation of the equations of frequency and of amplitudes can be found in most of the textbooks mentioned earlier, only the essential properties of the wave in a homogeneous, isotropic, elastic half-space are described here:

- The Rayleigh wave is connected to the free surface of the half-space.

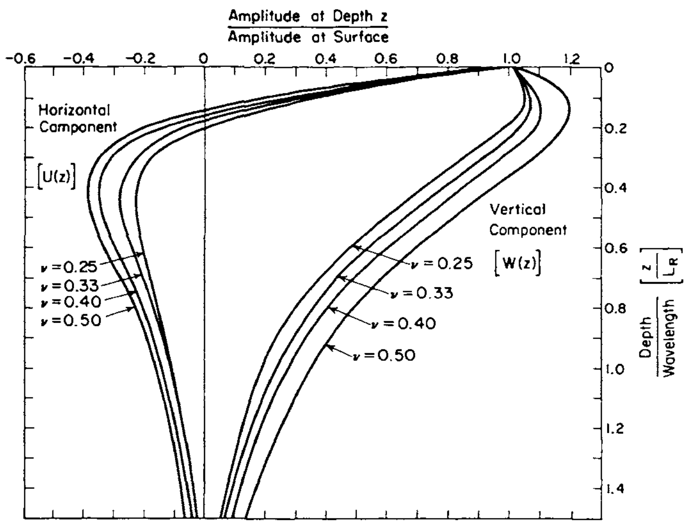


Fig. 5: Amplitude ratio vs. depth for Rayleigh wave (Richart et al., 1970)

- It consists of a vertical and a horizontal vibration component, the amplitudes of which quickly decay with depth, as may be seen in Fig. 5. Poisson's ratio is of little influence.
- At the surface, the horizontal component shows a value of about 0.6 to 0.8 of the vertical one. The "zone of penetration" of the R-wave is assumed to be one wave length L_R from the surface, because about 90 % of the wave energy is transferred within this zone.
- The velocity of propagation V_R is slightly smaller than V_S. It ranges between 87% (ν = 0) and 96 % (ν = 0.5) of V_s and attains a value of 0.92 V_s at ν = 0.25. In general one can write $V_R = c \cdot V_S$, with approximately $c = 1/(1.135 - 0.182\ \nu)$.
- The phase angle of W(z) is constant with depth; that of U(z) jumps by the amount of π at the depth where the amplitude vanishes ($z \approx 0.2\ L_R$).
- The components U(z) and W(z) are out of phase by $\pi/2$. The motion of a point at the surface during R-wave propagation is therefore a retrograde ellipse with the axes being vertical and horizontal.
- Material damping does not greatly affect the amplitude functions, but the phase angles are no longer constant with depth and their difference is not $\pm\ \pi/2$ (Caloi, 1948; Wolf/Obernhuber, 1982).
- The Rayleigh wave propagation is practically unaffected by a ground water table, except by the increase in soil density due to the pore water.

3.2 WAVE PROPAGATION FROM A FOUNDATION

In many cases of wave propagation problems, the wave source can be modeled as a rigid foundation located at the half-space surface and loaded by a harmonically varying force. The subsurface soil is considered for the present as a homogeneous, isotropic, linear elastic body.

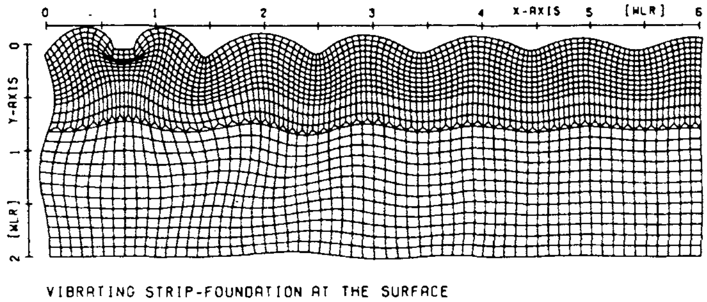


Fig. 6: Wave field due to a vertically oscillating strip foundation; FE-calculation (Haupt, 1978)

The oscillating foundation generates body (P- and S-) waves which radiate downwards into the interior of the half-space and to the side of the footing. At the surface, the Rayleigh wave with its characteristic properties develops quite soon from the body waves, because it represents the eigenmode of the half-space. The region close to the wave source, where the body waves are dominant, is called the near field. In contrast, in the so-called far field, the energy transport away from the source is governed by the Rayleigh wave in the zone near to the surface. The limit between the near and the far field is fluid; at small vibration sources it is often taken to be 1.0 to 1.5 L_R.

Fig. 6 shows the plane FE grid considered by Haupt (1978). The upper boundary represents the free surface of the half-space and at the lower boundary, the reflection-free dashpot boundary condition is applied. The lateral boundaries are fully wave transmitting. In this Figure, the distorted FE-grid is to be seen at any moment t = const. (flash light view). Close to the foundation, i.e. in the near field where only body waves exist, irregular deformations may be observed, but at 1 to 2 L_R distant from the source at the surface, the typical R-wave displacement characters are conspicuous. Fig. 7 depicts

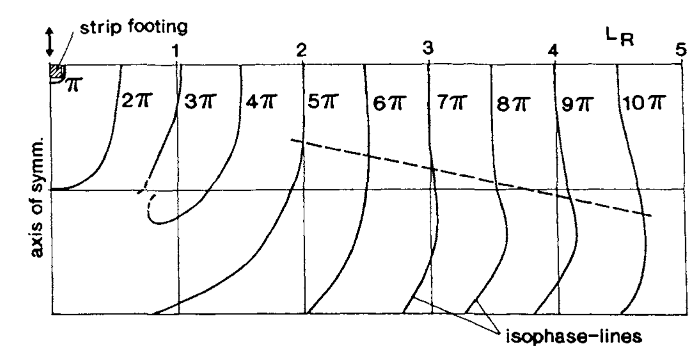
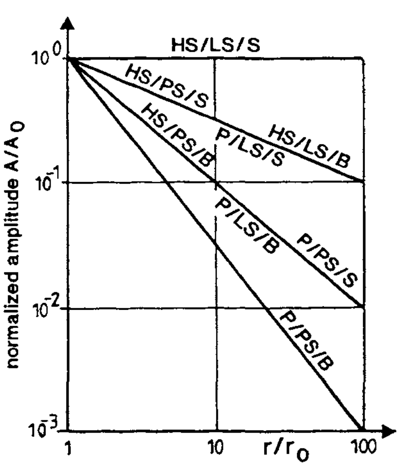


Fig. 7: Isophase-lines in the vicinity of a vertically oscillating strip foundation; from plane FE-calculations (Haupt, 1978)

the isophase-lines of the vertical vibration component of this displacement field. Along each curve the phase angle has a constant value of π, 2π ... $n\pi$. The direction of propagation of the waves (vertical component) is always perpendicular to these curves. Attention is drawn to the zones above and below the dashed line: while at the surface the isophase-lines are vertical and exactly $L_R/2$ distant from each other (as it should be), below the dashed line the distance is about 10% larger, which corresponds to the shear wave length. Thus, it seems that one can separate quite clearly the area of dominance of the Rayleigh wave and of the shear wave propagation.

In an axis-symmetrical system, the body waves propagate radially outward from the source along a hemispherical wave front. Therefore, their amplitude decreases approximately in proportion to the ratio of $1/r$ (r is the distance from the foundation) except along the surface of the half-space, where the amplitude decreases as $1/r^2$ (Ewing et al., 1957). The displacements of the Rayleigh wave in the far field are given by Bessel functions, which can be approximated by a harmonic function with an amplitude decrease as $1/\sqrt{r}$. This corresponds with the Rayleigh wave propagating radially outward along a cylindrical wave front. These geometric decay functions have nothing to do with the decrease of amplitudes due to the material damping.

Miller/Pursey (1955) have determined the energy content of the waves radiating from a vertically vibrating, circular energy source at the surface of the half-space. They found the energy partition among the three elastic waves to be 67% Rayleigh wave, 26% shear wave and only 7% compression wave. The fact that the Rayleigh wave transmits two thirds of the dynamic energy input of the foundation and, in addition, decays much more slowly with distance than the body waves, renders the Rayleigh wave most important in wave propagation problems.



Type of Vibration		HS		P	
Wave Source		PS	LS	PS	LS
Type of Wave	B	1.0	0.5	1.5	1.0
	S	0.5	0	1.0	0.5

PS = Point source
LS = Line source
B = Body wave
S = Surface wave

HS = Harmonic, steady-state; P = Pulse

Fig. 8: Amplitude decay of waves propagating from a wave source at the surface (EAD, 1992)

A steady-state, harmonically vibrating small foundation, as considered above, represents a special case. In reality, the wave amplitude decrease with distance depends essentially on three parameters: the type of the wave, the duration of its generation and the shape of the wave source. In the recommendations of the German Geotechnical Society (EAD, 1992) this has been taken into account in a simplified scheme shown in Fig. 8. The exponent n refers to the general wave equation

$$u_o(r) = u_o(r_o) \cdot (r/r_o)^{-n} \cdot e^{-\alpha(r-r_o)} \qquad (6)$$

with r_o = reference distance and $u_o(r_o)$ = amplitude at r_o. These recommendations are based on theoretical results as well as on practical experience, and they take roughly into account the influence of e.g. damping, layering and wave velocity increase with depth. The relatively strong decay of the peak values of body wave pulses can commonly be observed at vibrations from blasting. By regression analyses it was found that the peak value of the vibration velocity decreases as r^{-1} to $r^{-1.5}$ (Medearis, 1979).

With a vertically oscillating wave source at the surface, the Rayleigh wave shows its typical velocity of propagation at both the vertical and the horizontal displacement component. The same is the case with the vertical component if the waves are generated by a horizontally (in the direction of wave propagation) vibrating foundation. However, FE calculation results by Haupt (1985) show that this is not true for the horizontal wave component at the surface, in cases where it is generated by a steady-state, horizontally oscillating source. Fig. 9 depicts the amplitude and the phase angles depending on the distance for this case. The numbers 1, 2 and 3 refer to the surface and the levels immediately below the surface (0.1 and 0.2 L_R). The average slope of the phase angle curves as defined in Fig. 1 clearly indicates a propagation of the vibrations at compressional wave velocity. This is valid at least over a distance of about 8 L_R from the source. Moreover, at the surface, the amplitude shows an interference pattern which is only possible if there are both a horizontally propagating compressional wave and the Rayleigh wave interfering. A similar amplitude curve was found by Chouw et al. (1991).

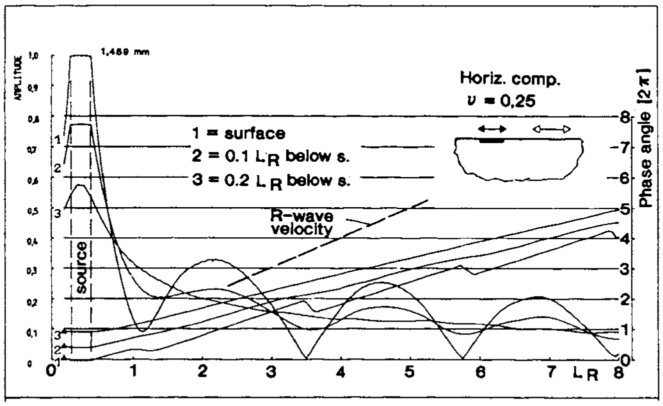


Fig. 9: Amplitude and phase angle at the surface, horizontal component with horizontally vibrating foundation (Haupt, 1985)

3.3 INCREASE OF WAVE VELOCITY WITH DEPTH

As has been shown in chapter 2.5, in the general case the shear modulus depends on the state of stress $\bar{\sigma}_0$ in the soil by a power law with the exponent m which usually is taken to be 0.5. If we consider a uniform ground with no structural inhomogeneities, the overburden pressure increases proportionally to the depth. Taking eq. (3) into account, this results in:

$$G \sim (\bar{\sigma}_0)^m;\ \bar{\sigma}_0 \sim z;\ \rightarrow G \sim z^m \rightarrow V_S \sim z^{m/2}, \quad (7)$$

where z = depth. Thus, even a completely uniform ground is inhomogeneous with respect to wave propagation. Therefore, in reality, the homogeneous half-space practically does not exist.

DISPERSION

By progressively decreasing the frequency of a steady-state harmonic wave source the Rayleigh wave length L_R is increased. Consequently, the zone of penetration reaches deeper into the half-space and the Rayleigh wave affects soil at a greater depth, which shows a higher wave velocity corresponding to eq. (7). Hence, due to the integrating property of the Rayleigh wave, its velocity of propagation is also increased. Thus, in a half-space with shear modulus increasing with depth, the surface wave is dispersive, i.e. the wave velocity V'_R depends on the frequency, with V_R increasing while f decreases. Since the amplitude functions with z are no longer those presented in Fig. 5, it is more correct to talk about the surface wave instead of the Rayleigh wave.

Different soil models have been developed to account for the increase of G(z). The power law described above (m/2 = 0.25) is primarily valid for non-cohesive soils. Gazetas (1982) has investigated this law with respect to wave propagation by varying the exponent within a wide range, as well as the degree of heterogeneity. He assumed a finite value of V_s at the surface, which may be representative for cohesive soils. A limiting case of this law is the shear modulus increasing linearly with depth, the so-called elastic Gibson's model (Gibson, 1967). Vrettos (1991) has approximated the power law with an expontential function, which has the advantage of a finite value of G at $z \rightarrow \infty$. He also showed that in this soil model, as well as the fundamental mode of the surface wave, higher modes also exist, which propagate with greater velocity. However, these modes are not of significance near the surface but become important in the interior of the half-space.

EQUIVALENT DEPTH

Whatever model is assumed, at a given frequency there is an effective depth z_{eq}, at which the shear modulus has a value which corresponds to the velocity V'_R. Hence, it can be stated:

$$V_R = c \cdot \sqrt{G(z_{eq})/\varrho}\ , \quad (8)$$

where $c = V_R/V_s$ is taken from chapter 3.1. The "sampling depth" z_{eq} is approximately a constant portion of the effective wave length L'_R. This relationship is used in the R-wave dispersion measurement. In this test, the wave velocity V'_R is determined at different discrete

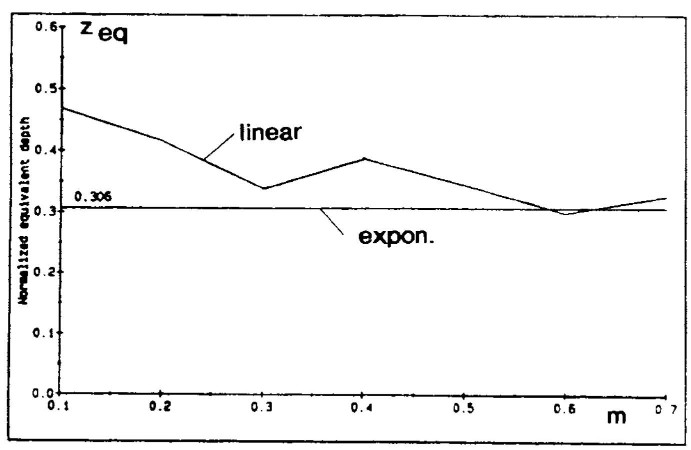


Fig. 10: Normalized equivialent depth for non-homogeneous half-space (Leung et al., 1991)

frequencies. This can be done by comparing the phase angles of a steady-state harmonic wave field at the surface at different distances from the source. Once L'_R (f) is known, the dispersion curve V'_R (f) can be calculated, from which V'_R (z_{eq}) is obtained. Hence, by use of eq. (8) this method provides a profile of the shear modulus with depth. The steady-state R-wave method represents a simple, quick way to obtain information about the dynamic properties of the near-surface ground.

During the past two decades, an empirical value of $z_{eq} = L'_R/2$ has been widely used for this equivalent depth (Richart et al., 1970). The half wave length rule has also been applied successfully to layered soils. However, is seems that this rule is valid primarily for grounds showing a small degree of heterogeneity. In his theoretical investigation, Gazetas (1982) found $z_{eq} = L'_R/2$ to be a good average value for power law models including the linear case, provided the degree of non-homogeneity was not large. On the other hand, there are some theoretical and experimental results which indicate that with a shear modulus increasing by a power law with m ≈ 0.5, z_{eq} is in the order of 0.3 L'_R (Heisey et al., 1982). For such a soil from FE calculations Haupt (1986) found a ratio z_{eq}/L'_R = 0.25. In theoretical investigations, Vrettos (1990) has shown that in his approximate soil model z_{eq}/L'_R is about 0.3 to 0.35. The number of 0.3 was strongly confirmed by Vrettos/Prange (1990), who carried out R-wave dispersion measurements on a ground consisting of sand and gravel. Leung et al. (1991) investigated the behaviour of waves by the plane BEM in a half-space with variation of shear modulus corresponding to Vrettos' model. They obtained z_{eq} to be 0.306, as is indicated by the horizontal line in Fig. 10. For a soil with a linearly varying shear modulus, the ratio z_{eq}/L'_R was in the range of 0.3 to 0.45, where the latter value corresponds to smaller degrees of heterogeneity (Fig. 10). Thus, it seems that with cohesive soils like stiff clay or soft rock, which can be modeled by the Gibson's soil with a small degree of non-homogeneity, the half-wave length rule applies quite well, whereas for cohesionless soils with a relatively small shear modulus at the surface, an average z_{eq} = 0.3 L'_R should be used.

BENDING OF BODY WAVES

In a half-space where wave velocity increases with depth, the body waves which radiate from a steady-state oscillating foundation at the surface to the interior of the half-space are bent continuously upwards and return back to the surface. There, a superposition occurs of these body waves and the surface wave, yielding a more or less di-

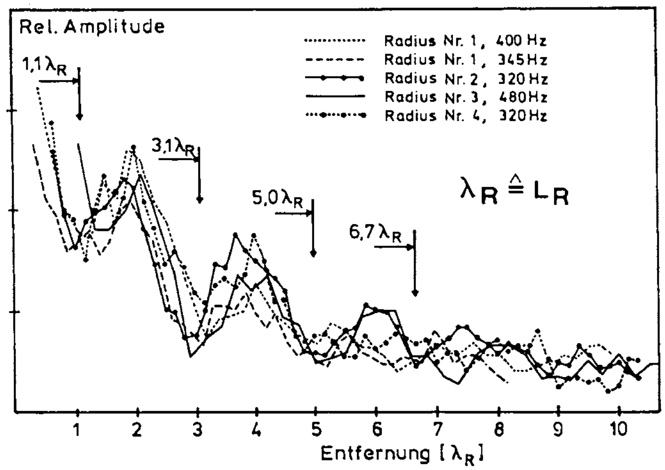


Fig. 11: Interference pattern at the surface due to non-homogeneity of the half-space; measurements (Haupt, 1986)

stinct interference pattern superimposed to the theoretical amplitude decay curve. This phenomenon can practically always be observed in reality (Barkan, 1962; Gaul/Plenge, 1992) and appeared in FE-calculations on an inhomogeneous half-space as well (Haupt, 1986). Measurements showing such irregular decay curves with distance, therefore, are not necessarily incorrect. During tests on the wave propagation at the surface of a very uniform sand body, Haupt (1986) measured these interference patterns which at different frequencies were not at all similar. However, after normalizing the distance on the respective measured wave lengths L'_R Fig. 11 was obtained, showing a very good coincidence of the curves. The writer has observed that in practice, the distance between the minimum points is often in the order of 1.5 to 2.0 L'_R. The bending of body waves may also be of importance in seismic investigation methods like refraction survey or cross-hole test (Gazetas, 1982).

3.4 LAYERED HALF-SPACE

Another kind of imhomogeneity of the half-space is layering. Any combination of abrupt wave velocity change with depth is possible, including stiff top layer over a softer half-space, as happens in permafrost regions. Therefore, simple solutions are not available. The case of a softer layer over a stiffer half-space is quite frequent in reality and has been subject of much research work. A strong situation of this type is given by one or more soil layers over bedrock, a problem which is of great importance for the evaluation of a site with respect to the seismic risk.

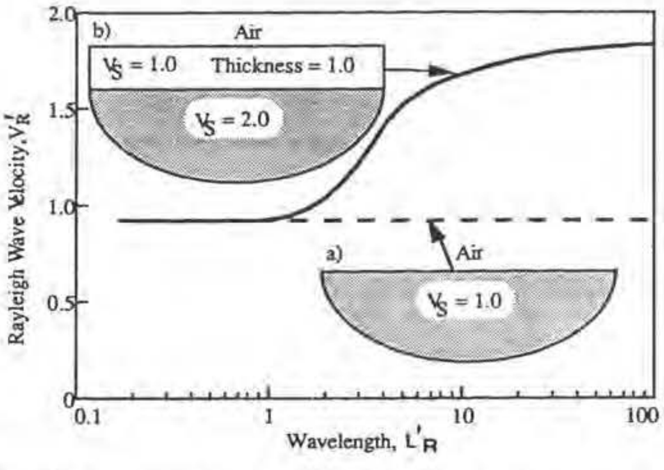


Fig. 12: Dispersion curves for plane Rayleigh waves
a) uniform half-space; b) soft layer over stiffer half-space (Stokoe et al. 1994)

If body waves, travelling from the interior of the half-space upwards, are transmitted from the bedrock to the soil layer, an amplification of the amplitudes takes place, depending on the eigenfrequencies and the eigenmodes of the layer. The resonance behaviour of the layer is strongly influenced by the material damping and also, to some extent, by the degree of non-homogeneity of the soil layer, as considered in the previous chapter (Gazetas, 1982). According to Wolf/Obernhuber (1982a, b) the angle of incidence of the body waves to the interface is an important parameter influencing the degree of amplification, as well as the propagation of waves at the surface of the layer. Erlingsson/Bodare (1992) have analyzed the resonance behaviour of a layer of soft clay over a bedrock by considering a material with the shear modulus depending on the dynamic strain. With this model they could explain the damage at a stadium structure due to an audience jumping during a rock concert.

SURFACE WAVES

In a layered half-space the surface wave is dispersive. It can well happen that the wave velocity does not increase monotonically with increasing wave length (Tokimatsu et al., 1992b; Rix, 1988). With such more complicated layer systems the simple concept of the effective depth cannot yield a detailed profile of the dynamic soil properties with depth, because the surface wave velocity is the result of an averaging process over the depth of penetration. However, it can be used to provide general information about subsurface conditions (Roësset et al., 1991). Fig. 12 shows the surface wave velocity depending on L'_R for the case of a soft layer over a stiff half-space. This is the fundamental mode. The wave velocity is that of the layer material, as long as L'_R does not exceed the thickness of the layer. In this case, there are quasi homogeneous half-space conditions for the Rayleigh wave. If the wave length increases beyond the depth of the interface, then V'_R also increases and approaches the value of V_R of the half-space material, but only at very large wave lengths. The top layer has obviously the greatest influence on V'_R. An example including the additional effect of the continuously increasing shear modulus in the top layer may be found at Gazetas (1991).

In order to analyze the wave propagation in layered soils in the last decade, the Spectral-Analysis-of Surface-Waves method (SASW) has been developed. A comprehensive annotated bibliography on this method is provided by Hiltunen/Gucunski (1994). In this method, the dispersion curve V'_R (f) is obtained by the spectral analysis of a pulse which is generated at the surface and monitored by two or more receivers at different distances from the source (Stokoe/Nazarian, 1985). Then a velocity profile is assumed and a theoretical dispersion curve is calculated for that profile. This curve is compared to the dispersion curve measured at the site, and in an iterative procedure the assumed profile is modified until the theoretically and experimentally obtained dispersion curves match best. SASW results have been verified by the cross-hole test and the agreement was found to be satisfactory (Woods/Stokoe, 1985; Hiltunen, 1988). An example from Stokoe et al. (1994) for a multiple layer ground is presented in Fig. 13. The calculation of the theoretical dispersion curve on the basis of the dynamic stiffness matrix by Kausel/Roësset (1981) yields an infinite number of solutions (roots) for the wave velocity,

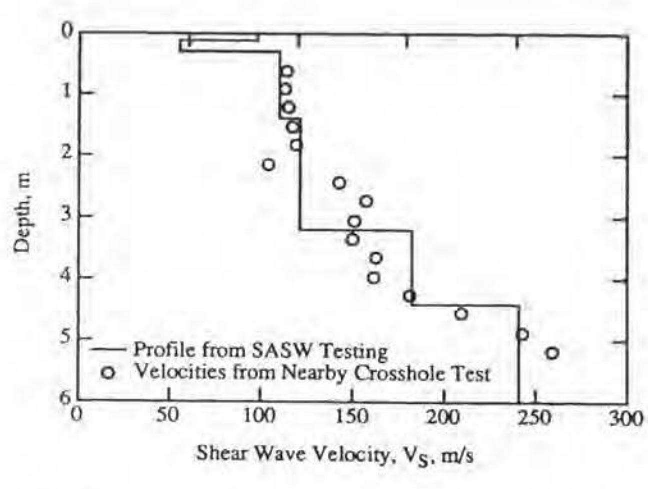


Fig. 13: Shear wave velocity profile, comparison of SASW test and cross-hole test (Stokoe et al., 1994)

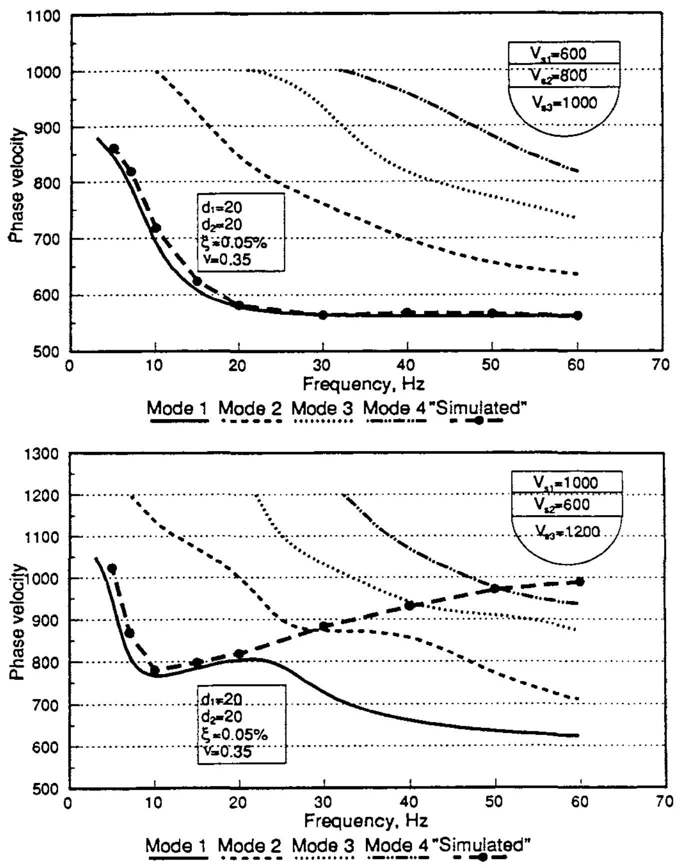


Fig. 14: Dispersion curves for plane Rayleigh waves and the "simulated" curve (Gucunski/Woods, 1991)
a) increasing shear modulus with depth
b) soft layer between stiffer layers

which are associated with the modes of the surface wave. In the present problem, only the first few modes are of interest and have to be superimposed in the right way (Tokimatsu et al., 1992a). Gucunski (1991) has shown that in the case of increasing layer stiffness with depth, only the first mode is significant, whereas in a complicated layer system higher modes are also involved. In Fig. 14 the "simulated" curve corresponds to the dispersion curve V'_R (f) as it would be measured at the site. This curve coincides well with the first mode in Fig. 14a, but with the soft layer between stiffer ones (Fig. 14b) successively higher modes come into play (Gucunski/Woods, 1991).

In the extreme case that an elastic layer is located on top of a rigid half-space, a critical frequency exists which corresponds to the lowest eigenfrequency of the layer $f_g = V/4H$, where H = thickness of the layer. Theoretically, at a lower frequency than f_g the wave cannot propagate through the layer and no energy is transferred away from the source. This phenomenon has given rise to a proposal for the isolation of vibrations in the ground (chapter 4.6). A horizontally polarized shear wave (SH) which travels in a layer over a half-space is called Love wave. As with the Rayleigh wave, it is dispersive and it shows the shear wave velocity of the layer material at high frequencies, provided this is smaller than V_s in the half-space. At low frequencies, the Love wave velocity approaches this latter velocity. If the ratio of the velocities is in the order of 3 or greater, it is possible to calculate the thickness of the layer and its shear wave velocity by an approximate theory from the dispersion curve measured at the site (Richart et al., 1970).

4 ISOLATION MEASURES

4.1 REDUCTION OF VIBRATION AMPLITUDE

The problem of isolating buildings or installations against shocks and steady-state waves in the ground is becoming more and more important. The reason, on the one hand, is the increased intensity of the vibrations generated by oscillating machine foundations or by highway or railway traffic. On the other hand, an increasing sensitivity of the objects affected by vibrations can be observed, as for example computers or highly accurate instruments for mechanical or optical measurement. Furthermore, today, people who live or work in the buildings are no longer prepared to tolerate the disturbance or inconvenience caused by vibrations. Hence, the intensity of vibrations due to waves travelling in the ground will often exceed the limit of tolerance if no precautions are taken.

There are two possibilities to reduce amplitudes at an object affected by vibrations. One way, referred to as the active method, comprises all measures at the source itself which are suitable to avoid the generation of vibrations, e.g. balancing a machine. On the other hand, the term passive isolation includes all those measures which reduce the vibrations at the object. Woods (1968) also applied these definitions to the screening of vibrations in the soil by trenches or other obstacles. Trenches located very near to the vibration source are called active isolation measures, whereas trenches remote from the source belong to the passive screening methods. This distinction is also linked with physical phenomena. Active isolation measures are placed in the near field of a wave source, i.e. at a maximum distance of about 1.5 -2 Rayleigh wave lengths. They are designed to shield the body waves which radiate from the source and are dominant within this immediate area. In contrast, the passive isolation measures are situated in the far field and are effective by screening mainly the Rayleigh wave. Their effectiveness does not depend on the distance from the vibrating footing, whereas this is the case with the active, i.e. near field barriers.

4.2 SURVEY OF PRESENT RESEARCH WORK

THEORETICAL INVESTIGATIONS

The propagation of waves in a non-homogeneous elastic body was the object of theoretical investigations by Knopoff (1959a, 1959b), who considered the diffraction of body waves at a rigid sphere. Dresen (1972) calculated the propagation of compression waves around a cylindrical cavity in a full-space. Achenbach (1976) considered the effect of a slit. The behaviour of body, as well as surface waves, at a corner or at an edge of the surface of an elastic body was investigated by approximate analytical techniques or experimentally by DeBremaecker (1958), Lapwood (1961), Kane/Spence (1963), Pilant et al. (1964), Hudson/Knopoff (1964) and Gangi (1967). Gilbert/Knopoff (1960) performed calculations on the propagation of a Rayleigh wave below and across a dam.

Calculations relating to the disturbance of a Rayleigh wave field by an inhomogeneity at or near to the half-space surface were reported by Hudson (1967). Aboudi (1973) considered the effect of a very narrow, deep barrier on a travelling Rayleigh wave pulse by the finite difference method. He stated an increasing isolation effect of the barrier with increasing depth and decreasing distance to the wave source. Analytical methods were applied by Diankui/Feng (1991) and Todorovska/Lee (1991), who considered the scattering of plane SH-waves from earthquakes at canyons of different shapes.

The FE method proved to be a powerful tool for the theoretical investigation of the wave isolation problem, as there are practically no restrictions on the dimensions, the position of the trench or barrier, and the infill material. Because of computer capacity, only two-dimensional problems were considered. The main difficulty at the FEM is the application of the appropriate conditions at the boundaries of the FE-grid, in order to avoid reflections. Waas (1972) studied the screening effect of open trenches on travelling SH-waves, using special types of elements to account for the radiation conditions at the boundary, as described by Lysmer/Waas (1972). His calculations are limited to the case of a relatively thin layer over an absolutely rigid base. Similar boundary elements were applied by Segol et al. (1978). They considered open and backfilled trenches of straight or trapeziodal cross section. However, since they also applied a rigid boundary at the base of the FE-grid, and in addition did not consider any internal material damping, their calculation results might be somewhat distorted by the reflection of the waves at the lower boundary. Haupt (1978 b) solved the problem of the non-reflecting lateral boundary by developing the influence-matrix-boundary condition, providing in addition the tool for very convenient and effective performance of the calculations. He considered infilled trenches of rectangular and irregular cross section at the surface and below it. The infill material properties were varied within a wide range, and active as well as passive isolation conditions were investigated (Haupt, 1978).

With the development of the boundary element method (BEM), the problem of isolating vibrations in the ground became of much greater interest to researchers in the field of soil dynamics, after attention had not been paid to this for some time. This numerical method is ideally suited to the investigation of elastic wave propagation in connection with geometrically complicated systems within a semi-infinite domain. Quite a number of papers have been published which will in part be considered in detail below. A comprehensive list of references concerning the physical and mathematical problems in question may be found in Beskos et al. (1986) and Leung et al. (1991). Recently, numerical methods were applied with regard to seismic wave scattering effects at surface canyons (Zhao/Valliappan, 1993) or embedded cavities (de Barros/Luco, 1993)

EXPERIMENTAL INVESTIGATIONS

In contrast to the relatively large number of theoretical studies, only a few experimental investigations have been reported. Barkan (1962) attributed the unsatisfactory screening efficiency of sheet pile walls, a factor often encountered, to the lack of knowledge of surface wave propagation in the presence of an obstacle, and therefore to the absence of a rational design procedure. Some successful applications of vibration isolation have been reported by McNeill et al. (1965) and by Neumeuer (1963), who had to deal with a slurry-filled trench and who clearly describes the difficulty of maintaining the infill material liquid.

Systematic tests on isolation measures have been performed by Dolling (1965, 1970) in an almost natural scale. He considered the screening effect of straight trenches up to 15 m in length and of 6 m in depth, which had to be stabilized with bentonite slurry. By varying the frequency of the steady-state vibration source, the Rayleigh wave length L_R adopted values between about 1.5 m and over 12 m.

The measurements carried out by Woods (1968) on a model scale included experiments on the active isolation as well as on the passive isolation. He clearly distinguished between the near field and the far (Rayleigh wave) field screening. The Rayleigh wave length varied between 0.34 m and about 0.7 m (1.1 ft to 2.25 ft) and the maximum depth of the trenches reached 1.2 m (4 ft). As the soil consisted of a silty sand, Woods succeeded in keeping these trenches open during the performance of the tests.

At open-air investigations in a natural soil, essential difficulties may be encountered. Among others, it is vital to find a test area with a very homogeneous subsurface soil and without a high-level water table. Furthermore, in order to keep the wave propagation conditions identical throughout the test performance, it may be necessary to protect the test area against rainfall or drying. Finally, the measured values must be guaranteed to be reproducible. Dolling has reported extensively on pre-tests to investigate wave propagation conditions and a good description of the measurement technique can be found in Woods (1968) and in Haupt (1978 a, 1981).

Indoor tests in a rectangular bin, 10 m x 10 m and a depth of 3 m, filled with uniformly densified sand, have been performed by Haupt (1981) to check the validity of his FE calculation results. The Rayleigh wave length was in the order of 0.20 - 0.5 m. He considered open trenches and rows of bore holes, but especially trenches with a stiff infill material corresponding to his theoretical investigations. Most of the tests were concerned with the far field, i.e. passive vibration isolation.

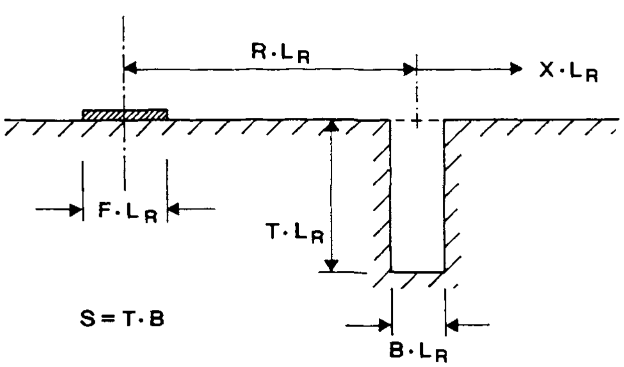


Fig. 15: Definition of geometric parameters

Experiments on an extreme model scale, with wave lengths of some inches (1 - 10 cm), were conducted by Woods et al. (1974), who introduced holographic interferometry to soil dynamics with great success. Using this method - up to that time only applied to measure the static or dynamic deformation of rigid elastic bodies - it is possible to observe the total steady-state harmonic wave field at the surface of a sand body in total, although restricted to a relatively small area of several square meters at the most. This new way of vibration monitoring was applied to wave barrier problems like rows of bore holes and open or infilled trenches. The same method was used by Plenge (1991) and Gaul/Plenge (1992) to observe the influence of buried solid obstacles on the surface wave field. They used Rayleigh wave lengths in the order of 0.2 to 1 m.

In the following comments, open trenches on the one hand and infilled trenches or other kinds of obstacles on the other hand are treated separately, because the wave propagation processes at the barriers, responsible for the reduction of the amplitudes behind them, are totally different. Since all theoretical investigations considered in the following chapters deal with a linear elastic problem, and this model applies more or less also to the experimental investigations, the normalization of the geometric parameters on the Rayleigh wave length is possible and reasonable. The normalized parameters defined in Fig. 15 will be used throughout the text to obtain comparability of different research results.

4.3 ISOLATION BY OPEN TRENCHES

Very often, the sources of man-made vibrations or shocks, as well as the objects affected, are located at the surface of the ground or not far below. Moreover, the greatest part of the dynamic energy spreading out from the source is propagated along the surface. Therefore, isolating measures of any kind have to be situated at the surface of the ground to yield best isolation effect. In this respect, definitely open trenches are superior to those measures based on the difference of material properties, because no energy can be transfered across them.

The effectiveness of a wave barrier is expressed by the amplitude reduction factor A_r, which is defined as the average value of the local normalized amplitude A(X) behind the open or infilled trench:

$$A(X) = \frac{\text{amplitude with barrier}}{\text{amplitude without barrier}}$$

In two-dimensional problems, A_r is calculated from the amplitude at the surface over a distance behind the barrier which, depending on the question, may include up to 10 L_R. To obtain a value most representative for the true three-dimensional problems he investigated, Woods (1968) defined the amplitude reduction factor A_{rd} as the average value within a certain area of the surface behind the trenches. In this way also the influence of local soil inhomogeneities was minimized. In the case of straight trenches, this area included half a circle behind the trench with the diameter of the trench length. Woods also required $A_{rd} \leq 0,25$ for a trench to be regarded as satisfactorily efficient. This criterion has been adopted by most later researchers.

ACTIVE ISOLATION

Woods (1968) performed active isolation with an angular trench around a point source at the surface generating steady-state vertical vibrations. He used full circle trenches but also segments of a circle, with a radius of not more than about one R-wave length ($R \leq 1$). The amplitude reduction factor A_{rd} was calculated over an area extending up to 10 L_R from the source.

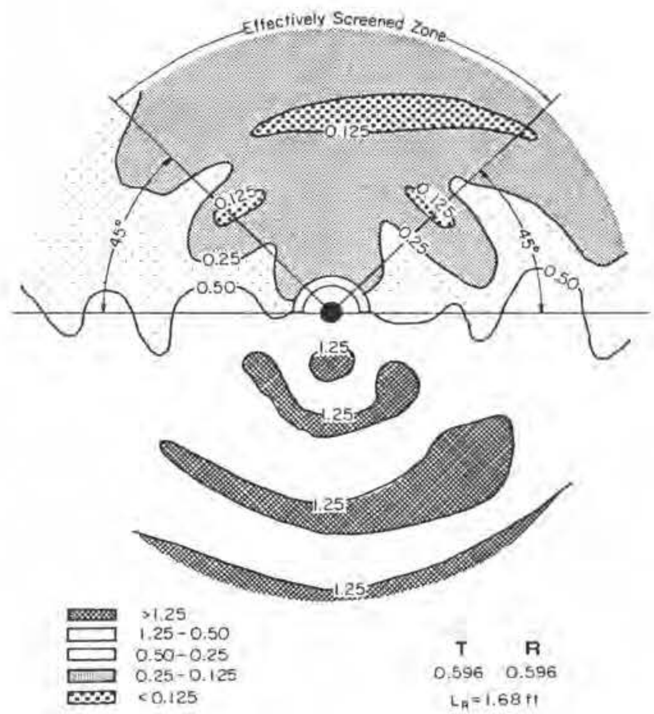


Fig. 16: Amplitude reduction factor contour diagram at active isolation (Woods, 1968)

By systematically varying the wave length, as well as the trench depth and the radius of the angular trench, he found the trench depth to be the most important parameter: A_{rd} decreases while T increases. Within the range considered, the distance R between the point source and the trench turned out to be an insignificant variable, as was the parameter B which was rather small (at maximum $B = 0.23$). The criterion for satisfactory screening efficiency noted above was found to be fulfilled by a trench depth equal to or greater than about 0.6 L_R ($T \geq 0.6$). This holds also for partial circle trenches, if the area considered is bounded at the sides by radial lines through points 45° from the ends of the trench (see Fig. 16). From this, it follows that 90° trenches are not effective. In Fig. 16, attention is drawn to the regular pattern of increased amplitudes at the side of the wave source opposite the trench, which is due to the positive superposition of the waves generated by the source and those reflected at the trench.

The same screening system was investigated theoretically by Ahmad et al. (1994) by application of the BEM as an axis-symmetrical and a true three-dimensional system. Introducing the soil parameters measured by Woods in the test field and using the same trench dimensions, they obtained qualitatively good agreement of the amplitude reduction factor A(X) calculated along a radial line. Also, in general, the criterion for the satisfactory efficiency of a trench with $T \geq 0.6$ was confirmed, though by applying the linear value A_r instead of A_{rd} and only for small diameters F of the wave source. There is also agreement that screening improves with increasing trench depth. However, the tendency is not uniform and systematic relations can hardly be observed.

On the basis of the large number of calculations, the authors have developed a numerical model allowing for the prognosis of the A_r-value, by taking into account the dimensionless geometrical parameters with quite sophisticated formulas. Applying these formulas to some of Woods' results, they obtain quite good agreement. Further calculations include, among other things, the effects of partially angular trenches and of rectangular footings. An important result of this investigation is that attention is drawn to the influence of the footing size on the active isolation effect of the trench. It is shown that the A_r-value increases distinctly if the radius of the footing increases, namely reaches values greater than 0.8 L_R up to 2.5 L_R. This applies especially for small clear distances between trench and vibrating foundation, which can be seen from Fig. 17. The authors believe that the reason for this footing size effect is to be seen in wave interference phenomena, which are due to the waves generated at different points of the extended foundation plate.

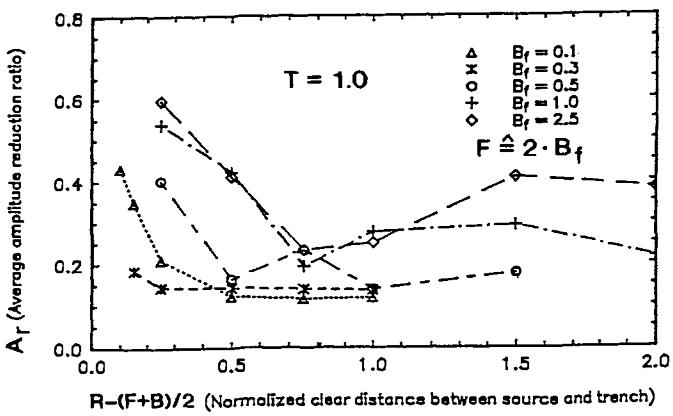


Fig. 17: Influence of normalized footing size on A_r for different barrier locations (Ahmad et al., 1994)

In fact, this interference pattern increases rapidly with increasing footing diameter. However, waves interfering jointly at the surface of the half space must propagate along this surface or near to it and must therefore be affected by the open trench to a somewhat similar extent. This explanation therefore seems unconvincing.

If the isolation effect of the trench is diminished, i.e. less wave energy is screened, then more energy must unavoidably pass below the trench, because it cannot pass across it. Any explanation of the above-mentioned effect must take this into account. If we consider a large, circular, vertically vibrating foundation plate inside an angular trench, as shown in Fig. 18, it is plausible that the actual source of waves spreading out to all directions is not located at the foundation plate but merely at the soil body, situated approximately at the depth of the trench bottom or even lower. In contrast, at the point source, under the same conditions, the origin of the waves is located at the surface. This model of the quasi origin of wave propagation can also explain the smaller effect of the active isolation at increasing values of υ (Ahmad et al., 1994)

An interesting result pointing in the same direction has been reported by Beskos et al. (1986), who investigated the isolation efficiency of open and infilled trenches by the BE method as a plane strain problem. A strip wave source (F = 0.5) was considered being remote from an open trench (T = 1.0; B = 0.1) by 1 L_R. The average reduction factor reaches the small value of A_r = 0,17. If, however, an identical, additional trench is located at the opposite side of the wave source, at the same distance, A_r rises to about 0.25.

The screening effect of a straight trench of finite length was investigated theoretically by Dasgupta et al. (1990), by applying the BEM as a true three-dimensional problem. The trench (T = 0.5; B = 0.1; length = 2 L_R) is placed symmetrically to the vibration source (F = 0.5) and 1 L_R remote from it. The amplitude reduction factor calculated over a length of about 1 L_R turns out to be A_R = 0.26, which corresponds quite well with the results of Beskos et al. (1986). The factor referring to the area behind the trench, as defined by Woods, is in this case A_{rd} = 0.31. This slightly higher value, which is to be expected, is due to the bending of waves at both ends of the trench.

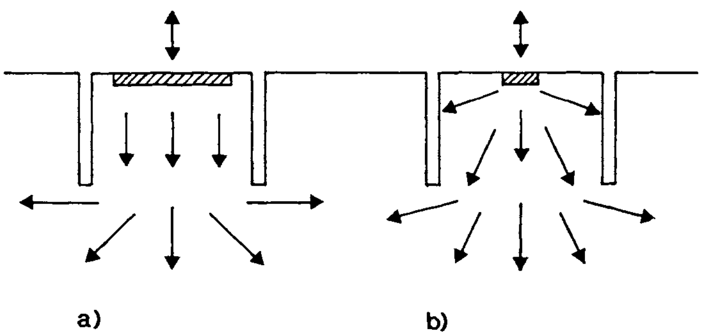


Fig. 18: Wave propagation at active isolation
a) large foundation plate b) point source

PASSIVE ISOLATION, EXPERIMENTS

The tests by Woods (1968) on the screening effect of trenches in the far field of a wave source have been carried out on straight trenches of different depth and length. As mentioned above, the reference area for the calculation of A_{rd} was a semi-circle behind the trench. A typical result of the measurements, in the form of the amplitude reduction factor contour diagram, is presented in Fig. 19. This diagram provides some interesting information. The greatest reduction in amplitude takes place directly behind the trench, in this case to the side of the center. Here there is something like a "wave shadow", which in other cases appears at the line of symmetry. In front of the trench and to the side, an increase in amplitude of at least 25 % can be observed, due to the reflection of the incoming Rayleigh waves at the open side of the trench and to interference phenomena. The bending of the surface waves at the ends of the trench yields an area of increased amplitude in this zone.

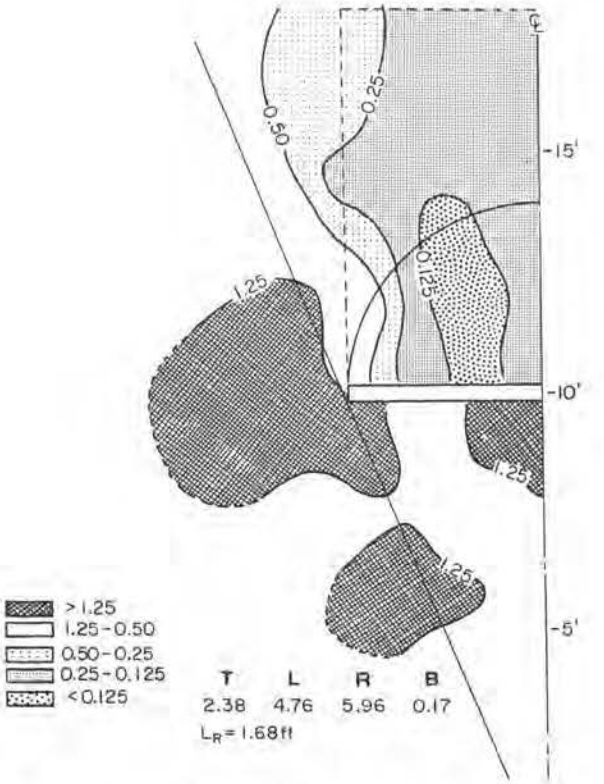


Fig. 19: Amplitude reduction factor contour diagram, passive isolation by a trench (Woods, 1968)

From the systematic variation of the trench geometry, its normalized depth T again turned out to be the parameter of most importance. An influence of the trench width B could not be established. Woods found that at the far field (surface wave) isolation, satisfactory screening efficiency ($A_{rd} \leq 0.25$) can be obtained only at trench depths of at least 1.2 - 1.4 L_R. This means that the passive isolation is less effective than the active isolation, which has been stated by Beskos et al. (1986) for open trenches, too. Some of the tests by Woods have been verified by Banerjee et al (1988) using a three dimensional BEM calculation. They considered exactly the same two layer subsurface soil which Woods had encountered at his test site with the dynamic soil properties derived from the measured wave velocities. The results, presented as A(X)-curves and as amplitude reduction factor contour diagrams, show an excellent coincidence, even in detail, with the experimental findings for the active as well as for the passive isolation case. This proves not only that the BEM can be a very powerful tool to investigate wave propagation problems, but also that the dynamic soil properties had been described correctly.

The tests performed by Dolling (1970) were related to slurry-filled trenches. Since shear waves cannot propagate through this fluid, these trenches, if situated in the far field of a wave source, are to be considered as open trenches rather than solid barriers. Woods et al. (1974) have proved this with respect to the vertical vibration component, by obtaining almost identical holographic patterns at an open trench and at the same trench filled with bentonite slurry. As an attempt to explain the mechanism of the isolation effect of trenches, Dolling developed a theory, based on the assumption that the energy of the Rayleigh wave in a region between the surface and the depth of the trench bottom is totally reflected at the front face of the trench. The Rayleigh wave energy below this depth passes beneath the trench, building a new Rayleigh wave at some distance, see Fig. 20a. Being derived from this two-dimensional concept of energy partition, the diagram in Fig. 20b shows the amplitude reduction factor A_r dependent on the normalized trench depth T. Poisson's ration ν does not play an important role, as can be seen. Attention is drawn to the fact that from this diagram a considerably better screening effect of deep open trenches as far field wave barriers is to be expected than has been established experimentally by Woods. Theoretically the criterion of $A_r \leq 0.25$ should be obtained at $T \approx 0.85$ (Poisson's ratio = 0.25). Actually, this relation has to be considered as a lower limit of the amplitude reduction factor at deep, narrow trenches, because no bending of the waves around the bottom of the trench is taken into account.

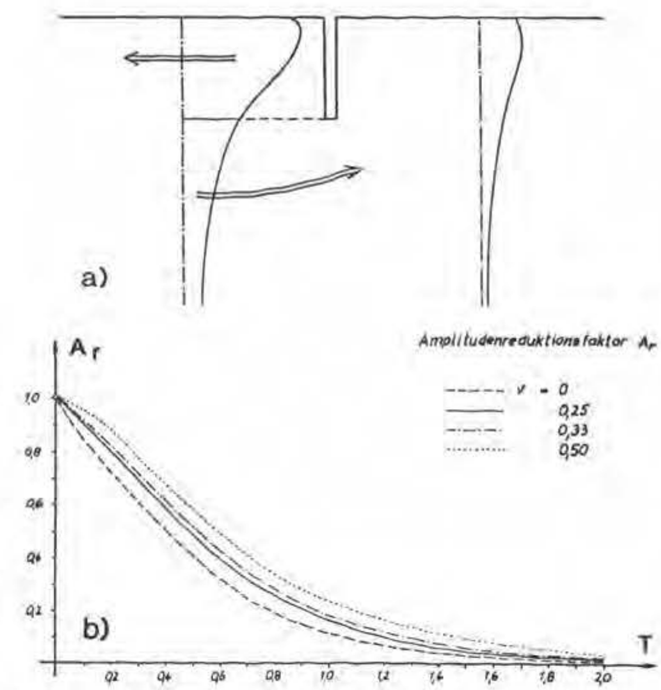


Fig. 20: Rayleigh wave energy partition by a trench
a) concept b) amplitude reduction factor (Dolling, 1970)

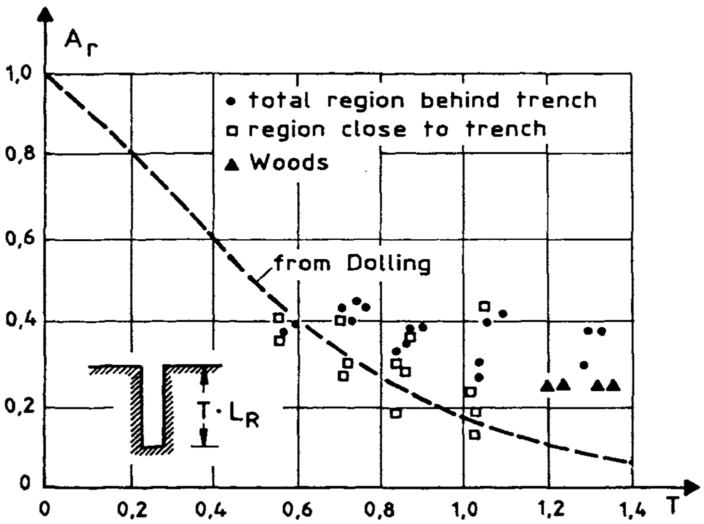


Fig. 21: Results of vibration isolation tests on open trenches (Haupt, 1981)

As far as the vertical vibration component is concerned, Dolling found his measurement results in fairly good agreement with this theory. This might be explained by the fact that the far field and near field conditions were not clearly separated. Most of the tests have been performed with a distance of 3 m between the wave source and the barrier. Therofore, except for the shortest wave lengths, near field conditions with distinctly better screening effects were encountered. In addition, many results refer to the zone of the wave shadow.

In the experimental investigations carried out by Haupt (1981) on open trenches on a model scale, the amplitude reduction of the harmonic vibration was measured at small intervals along the radius of symmetry of the system. He observed that the normalized amplitude was always much lower immediately behind the trench than at a greater distance. This nearby region obviously represents the wave shadow of the trench (Woods, 1968), which is to be derived from Dolling's theory, too. Therefore, the A_r-value had been calculated separately for this close region, which includes a length of about 1.5 L_R behind the trench. In a second calculation, that value was obtained as an average for the total region behind the trench, including a distance of about 3.5 L_R. The results of these measurements at narrow trenches ($B \leq 0.3$), in terms of A_r behind the barrier, are presented in Fig. 21 along with some results from Woods (1968) and the reference curve A_r (T) from Fig. 20b ($\nu = 0.25$).

Measurements on the screening effect of open trenches with a maximum depth of 2 m on transient vibrations were conducted on an only slightly reduced scale by Ciesielski/Zieba (1991). As a wave source, they used a falling weight. Normalized geometrical parameters cannot be ascertained, because due to the broad spectrum without a dominant frequency, there is no distinct surface wave length. The authors concluded from their test results that open trenches may have a good reducing effect to the maximum vibration values (A_r at minimum about 0.3) and that the effectiveness diminishes with an infill material like rubber scraps or styrofoam.

PASSIVE ISOLATION, THEORETICAL STUDIES

Unfortunately, Segol et al. (1978) did not present curves of the amplitude reduction factor A(X) or average values A_r. Therefore a comparison with other published research results is difficult. They did not distinguish between active and passive isolation, but they concur with other researchers in their observation that the isolation effect is unsatisfactory at T equal to/less than 0.6 L_R. Furthermore, a trench of trapezoidal cross- section with side slopes of about 60° at both sides is not inferior in screening efficiency to one with vertical sides.

At steady-state harmonic vibrations, the reflection of the incident Rayleigh wave energy at the trench generates a typical interference pattern, of which the maximum points appear at regular intervals of $L_R/2$. This pattern is generated by the superposition of two Rayleigh waves propagating in opposite directions. The first maximum

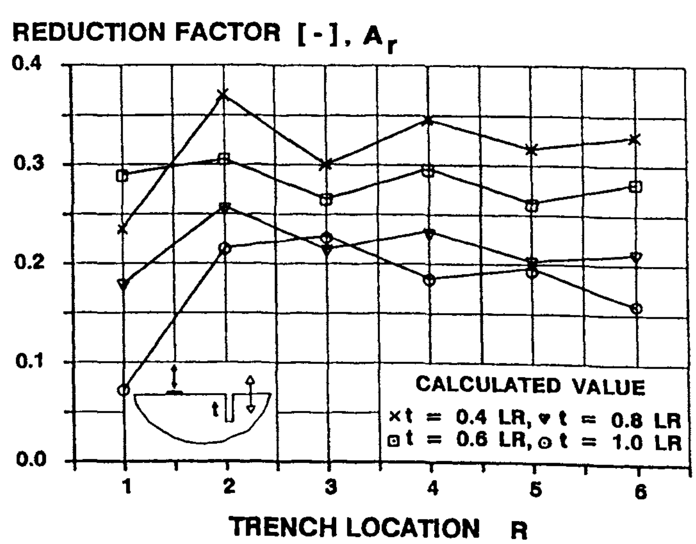


Fig. 22: Amplitude reduction factor at open trench depending on distance R (Chouw et al., 1991)

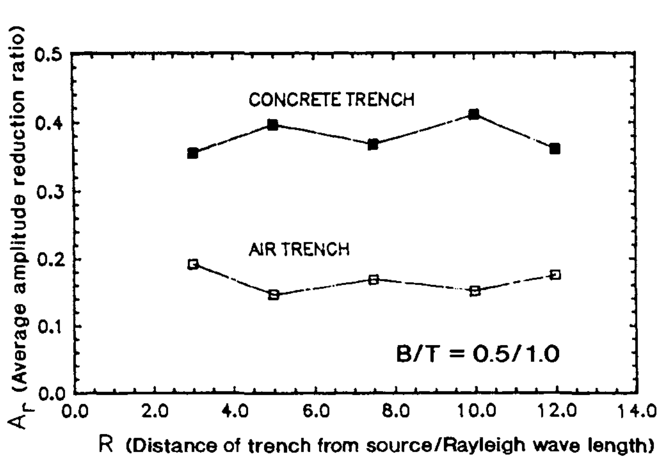


Fig. 23: Amplitude reduction factor depending on distance R (Ahmad et co., 1991)

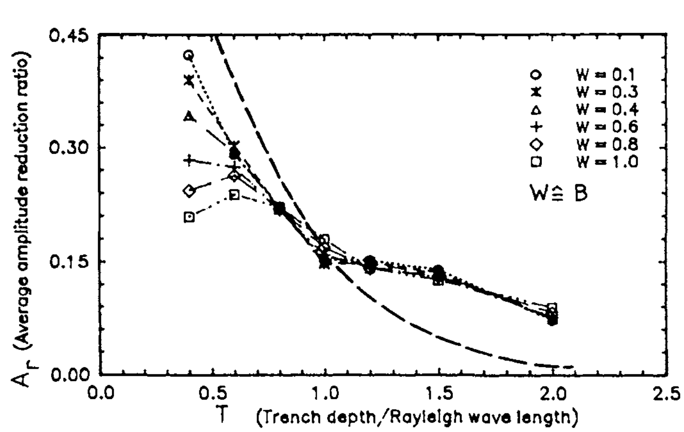


Fig. 25: Amplitude reduction factor at open trench depending on trench depth T (Ahmad et co., 1991)

point is located at the front face of the trench because this is an open end to the Rayleigh wave path (Segol et al., 1978; Beskos et al., 1986; Chouw/Schmid, 1991). A significant amplification of spectral energy was also observed by May/Bolt (1982), who investigated the effect of open trenches at the surface of a layered half-space on a travelling pulse by the plane FEM in the time domain. They found the spectral ratio distinctly reduced at frequencies related to wave lengths smaller than the trench depth.

The influence of the distance R between source and barrier is demonstrated in Figures 22 und 23. The decrease of A_r, i.e. the improvement of the isolation effect if the distance is reduced to the active isolation domain ($R \leq 2$), can clearly be seen in Fig. 22. At greater distances, this parameter has only minor influence, as is also shown in Fig. 23. Therefore, with values of R = 5, the following considerations refer to the purely passive case. A comparison of the essential results of the BEM calculations by Ahmad et co. and Beskos et al. is presented in Figures 24 and 25. The diagrams show A_r dependent on the normalized depth T, with the parameter of the curves being the trench width B. For purposes of comparison, the writer has inserted the reference curve A_r (T) from Fig. 20b (ν = 0.25) in these figures. First of all, the decisive influence of the trench depth is recognizable, being pronounced at small values. The diagrams coincide quite well in both the tendency and the numerical values. This has also been demonstrated by Ahmad et co. (1991) in a direct comparison of reduced amplitude curves. Some similar results are reported by LeHouédec et al. (1991).

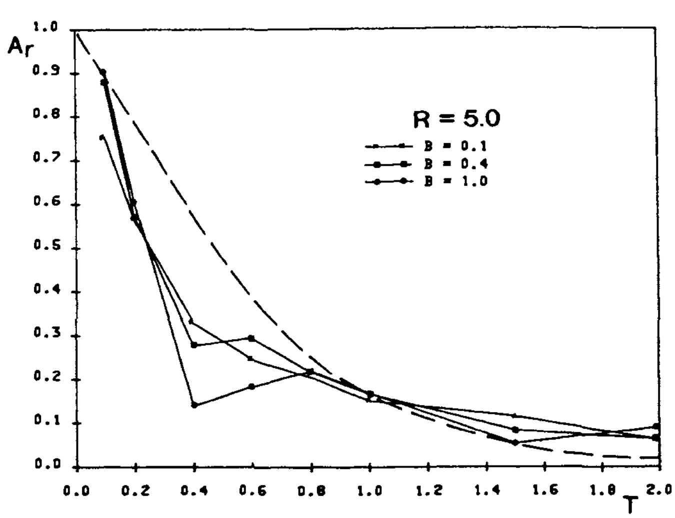


Fig. 24: Amplitude reduction factor at open trench depending on trench depth T (Beskos et al., 1986)

The criterion for satisfactory isolation effectiveness (A_r = 0.25) is reached in both cases at about T = 0.8, which is considerably smaller than had been obtained in the experiments. Beyond a range of T = 1.2 - 1.5, the improvement in screening effect is no longer very important, which agrees approximately with the findings of Woods (1968, 1974) and the results presented in Fig. 21. Except for shallow trenches, the influence of width B is small. The increase in isolation efficiency at normalized trench widths in the range of $0.4 \leq T \leq 0.6$ (see both figures) is explained by Ahmad et co. by the combined action of two wave propagation processes:

- At the front face of the trench, part of the R-wave energy is reflected.
- At the lower corners of the trench, a mode conversion, i.e. a transformation of the Rayleigh wave to body waves takes place, which contributes to the

isolation effect by radiating energy to the interior of the half-space.

For shallow trenches, a significant portion of Rayleigh wave energy is allowed to pass below the trench and the relative contribution of this mode conversion effect is high, especially if the opening is large enough to permit a Rayleigh wave to build up again at the bottom. For deep trenches, the reflection of the major part of the incident Rayleigh wave energy is what results in the low values of A_r; thus, the width effect becomes negligible. At very shallow openings ($T \leq 0.2$), however, the wave transformation effect also seems to vanish. The wave conversion at the trench bottom could possibly be the essential reason for the fact that at small trench depths the isolation effect from BEM calculations is considerably better than should be expected from the approximate theory by Dolling. At deep trenches, however, besides the R-wave reflection, in addition, some wave bending process must take place at the bottom of the trench, because otherwise the reduced isolation effectiveness, compared with the reference curve, cannot be explained. More information could probably be obtained by closer consideration of the wave field inside the half-space.

The influence of Poisson's ratio has been considered at the two values $\nu = 0.3$ and 0.48 (Ahmad et co., 1991). The results were found to be somewhat unsystematic and no general conclusion can be drawn from them. For practical purposes the influence of ν can be ignored. From their calculations the same authors have developed a simplified model for small open trenches ($B = 0.1 - 0.3$), at which the width may be ignored. The best-fit curve yields the relation

$$A_r(T) = {}^1/_6 \, T^{-1,07} \tag{9}$$

For the design of wide, shallow trenches, one is referred to Fig. 25.

The screening effect of an open trench to a non steady-state, i.e. a transient wave field due to an impact at the surface, was investigated by Emad/Manolis (1985), v. Estorff et al. (1990) and Beskos et al. (1986), by observing the displacement function with time at a point in front of the trench and at points behind it. No systematic relationship between the isolation effect and the trench depth or other geometrical parameters could be established; only some tendencies were to be observed (v. Estorff/Antes, 1991). However, since transient vibration problems are frequent in reality, further investigations into this direction will be worth while.

4.4 ISOLATION BY SOLID BARRIERS

In most cases, the researchers dealing with solid wave barriers have considered concrete as infill material for the trenches or for subsurface obstacles. They chose the wave length L_{RB} of this material being five times longer than that of the surrounding soil ($L_{RB}/L_R = 5$; $E_B/E = 34.7$; $\varrho_B/\varrho = 1.34$). This is a reasonable assumption in line with practical experience. Accordingly, unless otherwise stated, the following comments refer to concrete as infill material.

In the investigations already mentioned on stiff material wave barriers, which Haupt (1978, 1978a), conducted, using the FE method, the normalized dimensions T, B and R with reference to Fig. 15 have been varied systematically. Furthermore, he considered irregular cross-sectional shape of the barriers and different infill materials, including those with a smaller wave length than in the host material. His theoretical results were confirmed

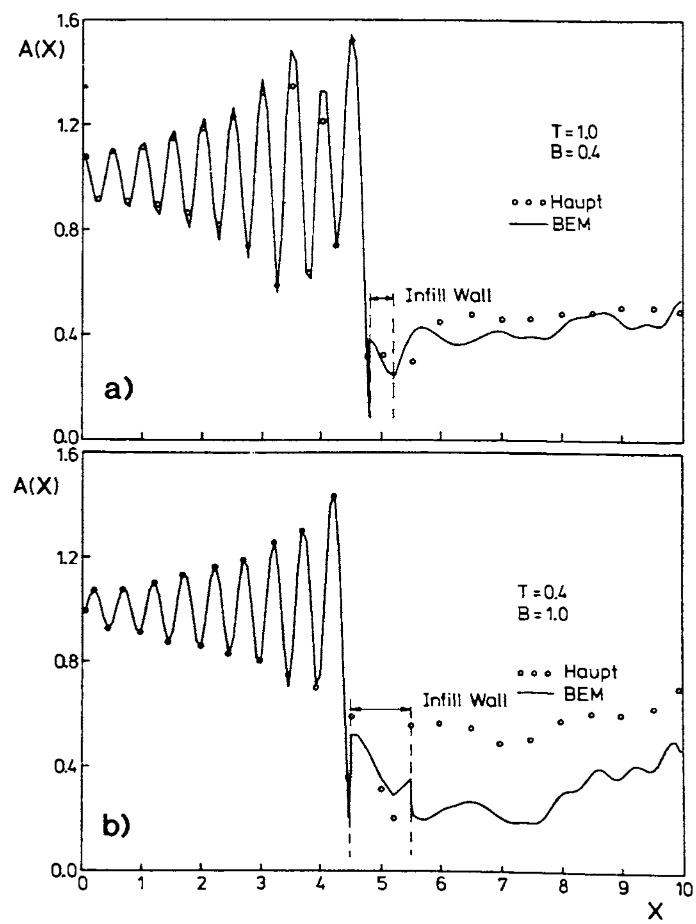


Fig. 26: Comparison of BEM and FEM calculations, infilled trenches (Beskos et al., 1986)

quite convincingly by accompanying model tests. Other systematic experiments on the influence of the abovementioned parameters are not known to the writer. The measurements by Woods (1974) are of a more qualitative nature. The research work by Ahmad et co. (1991), as far as it is concerned with infilled trenches, represents essentially a theoretical verification of Haupt's calculations, but this time performed by the BEM. They obtained essentially affirmative and only a few minor divergent results. Chouw et al. (1991) and Beskos et al. (1986) performed some exemplary calculations on this subject.

PARAMETERS OF INFLUENCE

In the case of infilled trenches, considerably better screening effect is also encountered with active isolation than with passive. Considering narrow, deep concrete walls (B = 0.1 and 0.2; T = 0.5 - 1.83) at distances in the range of R = 0.2 to R = 3.8 (F = 0.2), Haupt found the active case applying up to a distance of about R = 2. Beyond this value, that parameter is no longer important, which can be seen from Figures 22 and 23 and which was also confirmed by Beskos et al. The theoretical results concerning the better active isolation efficiency are strongly supported by the model tests (Haupt, 1978, 1981), as may be seen from the following data:

normalized cross-sectional area S	amplitude reduction factor A_r passive isolation	active isolation
0.38	0.47	0.22
0.19	0.73	0.49

In the case of soft material (L_{RB}/L_R = 0.5) infilling narrow trenches, active isolation does not perform better than passive. On the contrary, at very small distances the normalized average amplitude increases and may even reach values $A_r > 1$ (Haupt, 1978a).

The direct comparison of identical cases investigated by different calculation methods is always of special interest. Fig. 26 shows the normalized amplitude A(X) in front of and behind the barrier over a range of 10 L_R for two identical cases, once computed by the BEM and once by the FEM (Haupt, 1977). Firstly the distinct interference pattern in front of the trench is conspicuous, due to the Rayleigh wave energy reflection at the stiff body. The periodic distance of the maximum points is again $L_R/2$, as should be expected. Contrary to the open trench this time, it is a minimum point which is located directly at the front interface of the barrier, because the stiff con-

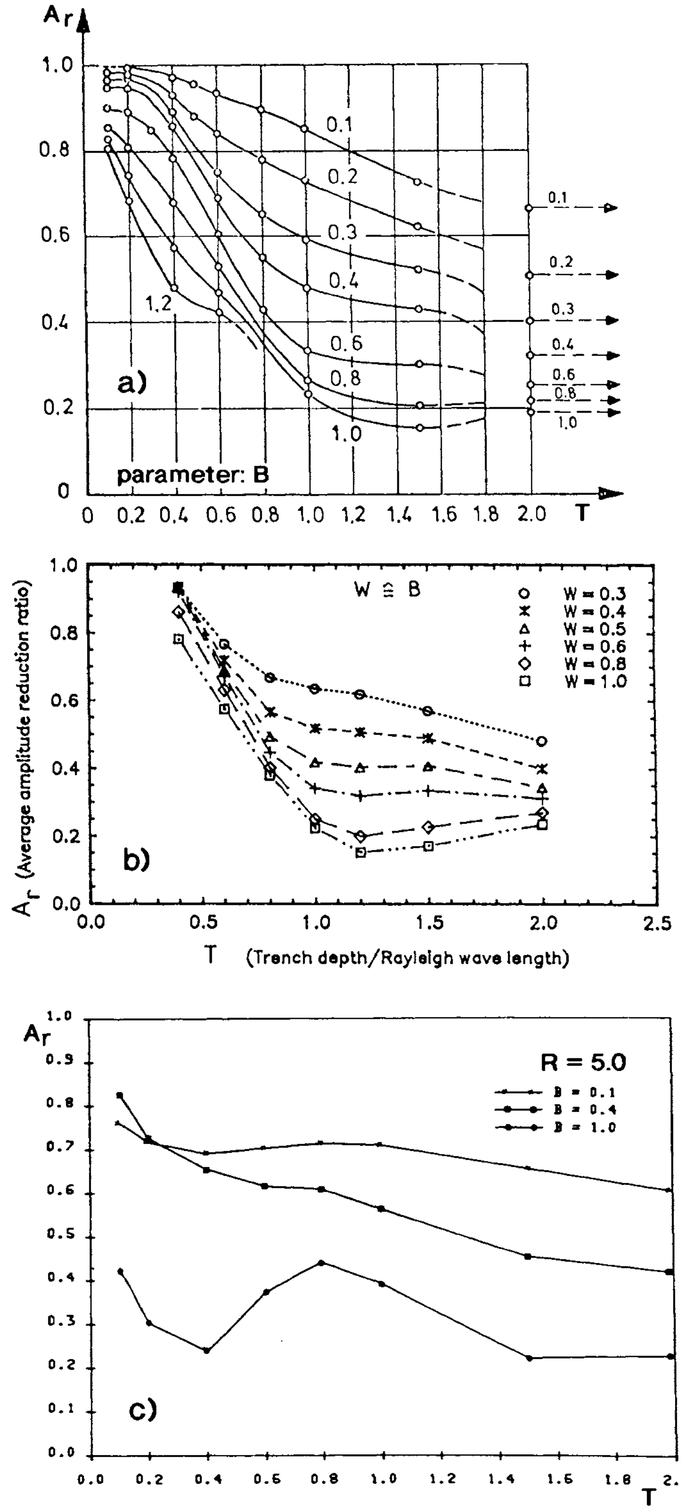


Fig. 27: A_r depending on T, infilled trenches
a) Haupt (1978) b) Ahmad et co. (1991)
c) Beskos et al. (1986)

crete wall impedes the displacements of the surface wave. At deep, narrow obstacles (Fig. 26a), the comparison yields a remarkable correspondance of the curves, even in details. However, in the case of a large, shallow concrete slab (Fig. 26b) while the interference pattern shows a perfect coincidence, the BEM-calculation reveals a distinctly better isolation effectiveness behind the trench.

Almost all researchers who have worked on the present topic have stated that the infilled trenches show a much smaller screening effect than the open ones. This is very evident in Fig. 23. Another important result quoted in most of the reports is the fact that not only the depth T of the barrier is an essential parameter, but also the width B, and this is not only valid for wide, shallow obstacles. In Fig. 27 the results of different investigations are compared with respect to these two parameters. It can be seen that the curves in Fig. 27a and 27b are in quite good agreement throughout, with respect not only to the tendency but also to the numerical values. The results presented in Fig. 27c, however, show a significant discrepancy from the first-mentioned ones concerning both tendency and values, the latter especially for obstacles of small depth. This corresponds to the difference in effectiveness of the barrier to be observed in Fig. 26b).

NORMALIZED CROSS-SECTIONAL AREA

At stiff obstacles, it has been found that the decisive parameter governing isolation efficiency is the product of width and depth, which is the normalized cross-sectional area:

$$S = B \cdot T.$$

From Fig. 28, presented by Haupt (1977, 1978) the conclusion can be drawn that the A_T-value depends more or less only on this parameter, i.e. the individual cross-sectional shape is eliminated. Thus, it is relatively unimportant for the magnitude of the amplitude reduction factor if a wide, shallow slab, a deep, thin wall or a body of more quadratic cross-sectional shape is considered, as long as these barriers have the same cross-sectional area. In view of the idealized system at the computations, the simple relation A_T (S) with a spread of about ± 20 % was regarded as precise enough for the design of concrete isolation measures in reality. LeHouédec et al. (1991) presented results, which agree quite well with those of Haupt, with both the tendency and the numerical values. A more detailed investigation on the influence of the actual shape of rectangular barriers, i.e. of the T/B ratio, performed by Ahmad et co. (1991), seems to result in a considerable dependency of the A_T-value on this parameter. However, if the cases of extreme dimensions (B > 1.0; T > 1.2) are omitted, the shape factor does not exceed the range of 1.0 to 1.4, which

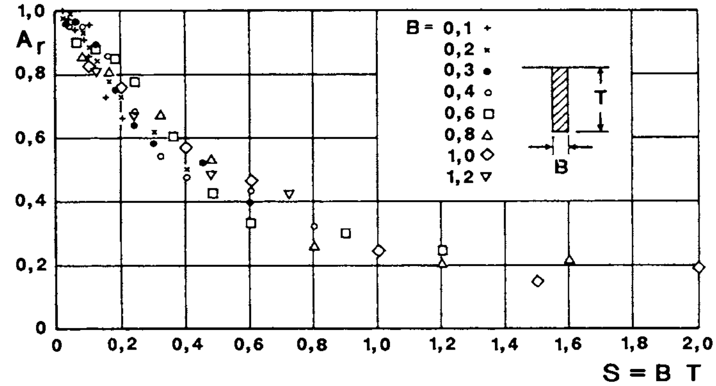


Fig. 28: Amplitude reduction factor depending on normalized cross-sectional area S (Haupt, 1978)

corresponds to a range of tolerance of about ± 20 %, too. The only exceptions are small, shallow obstacles, which apparently perform worse than has been found by Haupt (Fig. 28).

The present results might suggest that the mass of the obstacle is the essential parameter for the screening effect, because it is proportional to S. But that is not true. The stiffness of the barrier and the difference in wave velocity are the most important factors (Haupt, 1978). The performance of a deep, narrow barrier may shortly be explained by aid of Fig. 29a. The amplitude of the vertical vibration of the infilled trench, as well as the phase angle, are fairly constant with depth, which implies

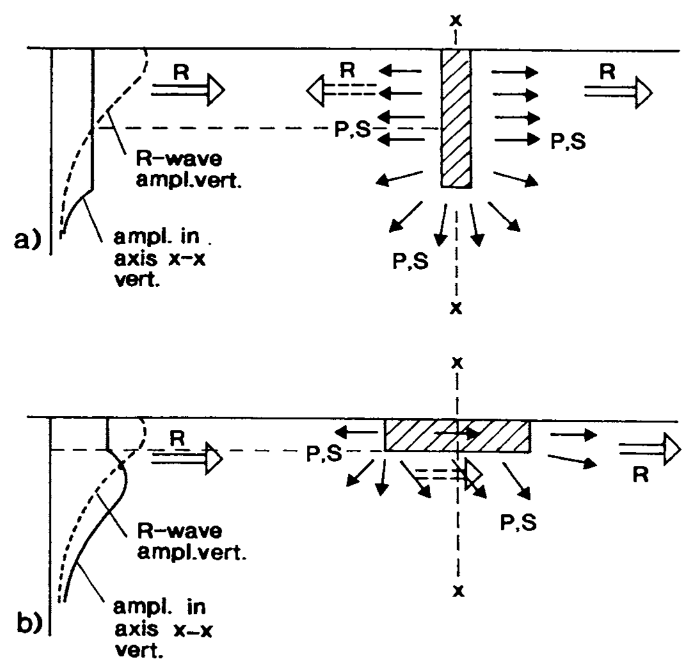


Fig. 29: Wave propagation processes at solid barriers

a vertical rigid body type vibration. In the upper part, the concrete wall impedes the free field Rayleigh wave displacement, resulting in the reflection of body waves, which quickly develop into a Rayleigh wave propagating backwards. The lower end of the solid barrier reaches down to a depth where the free field Rayleigh wave has only a very small amplitude. Thus, the lower portion of the obstacle acts like a new wave source, sending body waves into the interior of the half-space. At the rear side of the concrete wall, body waves are generated over the total depth of the barrier, developing again the Rayleigh wave at the surface. This agrees with Woods (1974), who had found a solid mortar barrier appearing to act like a new source of plane waves. Thus, the isolation effect of a deep rigid wall in the ground is based partly on the reflection of the Rayleigh wave energy and partly on its transmission to the interior of the half-space.

However, in this connection, the horizontal Rayleigh wave component should also be considered. At narrow walls, it generates bending vibrations of the barrier. The favorable effect of the increase of the obstacle width is possibly due to the way this bending vibration is impeded by the increased bending stiffness. There is a lack of reliable knowledge on this matter.

At a wide, shallow concrete slab, the wave propagation processes are totally different, Fig. 29b. The reflection in front of the body does not play this important role. At the edges of the rigid body and at its bottom face, mode conversion from the Rayleigh wave to body waves occurs, due to the impediment of the free Rayleigh wave field displacements. There is, however, a third phenomenon, which might be the most important one. That part of the Rayleigh wave energy transmitted through the concrete body to the zone of screening passes with a much higher wave velocity within the slab. As a result, the waves propagating through the barrier and under it in the soil are to some extent out of phase at the end of the slab. This results in partial destructive interference behind the rigid obstacle.

The wave propagation phenomena, demonstrated at the two barriers of extreme cross-sectional shape, combined in a complex fashion, determine the isolation effectiveness of solid obstacles in a way that the cross-sectional area is the most important parameter. This is also fairly true for totally irregular cross-sectional shapes or inclined walls (Haupt, 1978a). The correlations described above have been established on the basis of theoretical investigations. They are confirmed strongly by model tests performed on this subject by Haupt (1981), as may be seen from Fig. 30. The two dashed curves indicate approximately the band of the theoretical results for the far field isolation (Fig. 28). With the exception of three barriers with a higher wave velocity ratio which performed better, and the two cases of active isolation, all measured values of A_r are located within the band. By carrying out extensive Resonant-Column-tests it was proven that the infill material of the trench had practically the same dynamic material properties as have been used in the calculations: $\varrho_B/\varrho = 1.34$; $L_{RB}/L_R \approx 5.0$. For comparison, in the three cases mentioned above, the wave length ratio was chosen to be about 8.

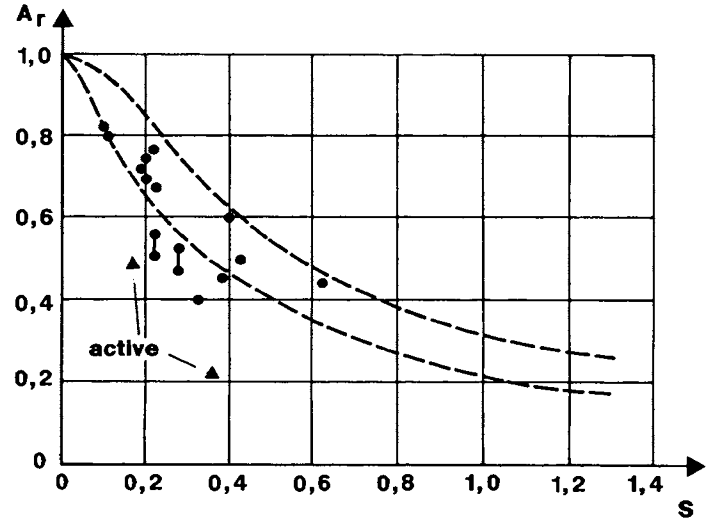


Fig. 30: Amplitude reduction factor from tests on solid barriers (Haupt, 1978)

OTHER PARAMETERS OF INFLUENCE

Although in reality one would likely use concrete as trench infill material, it is worth while considering the performance of other materials. Fig. 31 from Al-Hussaini/Ahmad (1991b) shows clearly the increase in isolation efficiency with the increasing ratio of wave velocity. For the range of geometrical parameters considered, the effect is always the same: up to a velocity ratio of about 3, there is a high influence on A_r, but from a ratio of about 5 on, only little better performance of the barriers is to be observed. This relation strongly supports the physical explanation of the isolation effect outlined above. Obviously a five times higher wave velocity of the barrier material is sufficient to produce its relative rigid body type motion. This offers the explanation for the often unexpected small and unsatisfactory isolation effectiveness of sheet pile walls, measured by Woods (1968) and by Haupt (1981) respectively. Though steel yields a higher velocity ratio, the efficiency of these barriers is not significantly better than a concrete core wall of small normalized cross-sectional area.

Sometimes the proposition is made to use soft infill material, i.e. material with a lower wave velocity than the soil, for instance rubber foam. Due to the absence of the rigid body vibration, barriers of this material are not able to transfer wave energy from the surface downwards to the inner part of the half-space. Therefore, deep, narrow walls are only effective by the partial reflection of the incident Rayleigh wave at the vertical material interfaces. Wide, shallow obstacles, however, can have a good screening efficiency, because of the partial destructive interference of the waves passing through and below the body (LeHouédec/Malek, 1991; Haupt 1978). This phenomenon only depends on the wave velocity difference, and not on the actual value of the wave velocity of the barrier material.

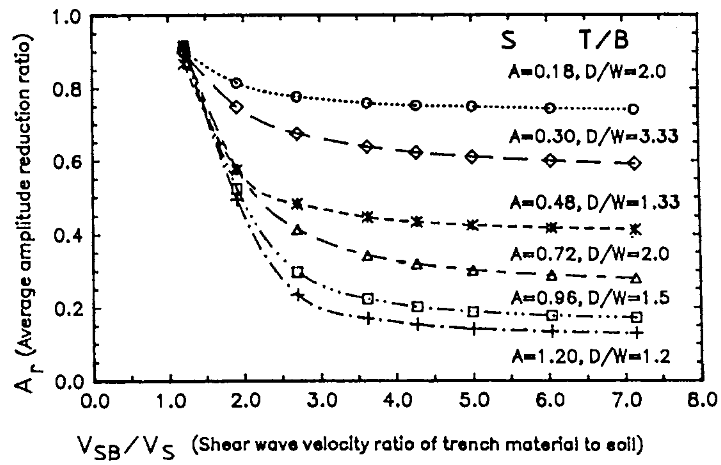


Fig. 31: Amplitude reduction factor depending on wave velocity ratio (Al-Hussaini/Ahmad, 1991b)

Very interesting results were obtained from investigations on barriers located below the surface. As may be seen in Fig. 32, the isolation effectiveness is very good, i.e. A_r is small as long as the depth of the upper face of the concrete obstacle, with dimensions of L_R by L_R, is less or equal to 0.4 L_R. The actual depth has practically no influence on the isolation effect. The A_r-value at τ = 0 corresponds to Fig. 27a. On the other hand, if τ reaches a value of 0.6 L_R or more, the isolation effect vanishes almost totally. It can be shown that the amplitude of the vertical vibration at the surface of the half-space above the obstacle in the cases τ = 0.5 and τ = 0.6 is increased as compared to the free field amplitude (Prange/ Haupt, 1974). An explanation for this whole

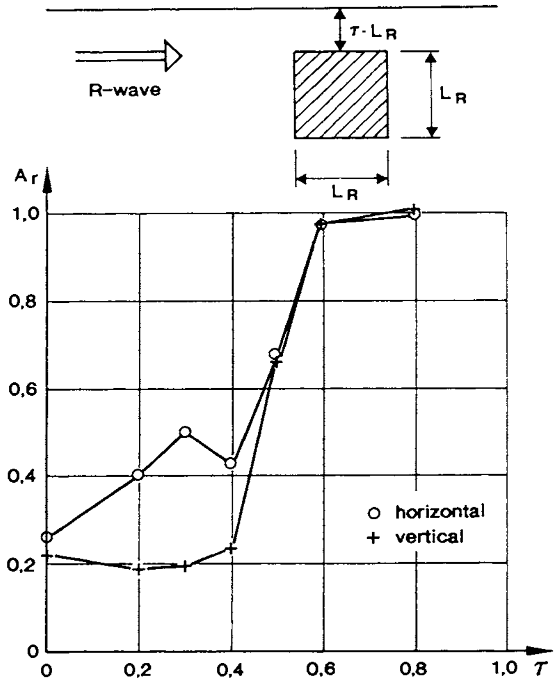


Fig. 32: Isolation effect of barrier below the surface, plane FE calculation (Prange/Haupt, 1974)

phenomenon is not known to the writer. If the effect were connected to the critical layer depth (chapter 3.4), it should occur at $\tau \approx 0.25$.

The influence of an oblique incidence of a surface wave on deep infilled trenches was investigated by Lee/Its (1994) in an analytical approach. They considered an infinitely deep trench of a width equal to 0.1 L_R, infilled once with a high velocity material representing concrete (L_{RB}/L_R = 2.7), and once with bentonite slurry as a low velocity material yielding L_{RB}/L_R = 0.4. The angle of incidence Θ_1 refers to a line perpendicular to the trench axis, hence the cases of plane strain dealt with in the foregoing correspond to Θ_1 = 0. The calculation results are presented in diagram Fig. 33, where the transmission coefficient is equivalent to the amplitude reduction factor A_r. The high velocity material shows a monotonic decrease of A_r with increasing angle. At the low velocity material, however, at an angle of incidence of about 67° the isolation effectiveness vanishes totally. The sharp descent of A_r at Θ_1 further increasing is explained by the authors by the significant role of the second interface (slurry-soil) due to its critical angle. At normal incidence of the waves (Θ_1 = 0) A_r for the high velocity infill material corresponds fairly well with other published results. The low velocity material, however, performs distinctly better than should be expected from FEM calculation results.

DESIGN MODELS

Again Ahmad et co. (1991) have developed a simplified method to determine the performance of solid barriers, based on a detailed statistical evaluation of their calculation results. The amplitude reduction factor is obtained as $A_r = I_s \cdot I_v \cdot I_d \cdot I_a$, where I_s = shape factor, to be read from the appropriate diagram. I_v = velocity factor, I_d = density factor, I_a = area factor are all functions of S which emphasizes the dominant role of this parameter. An analogue model was elaborated for the horizontal vibration isolation. Concerning the applicability of these models provided by the authors for all kinds of rectangular barriers, we should bear in mind the following considerations: The investigations on which these models are based are confined to the ideal situation of the homogeneous half-space. What is not taken into account is that in reality

- we often have some layering in the ground, very often water table (sometimes with changing level) and other natural inhomogeneities; there is in any case no homogeneous half-space with respect to wave propagation due to increase of the overburden pressure;
- there may be other buildings or installations in the neighbourhood disturbing the wave field;
- the wave source is usually not simply shaped (point or line source) but often a number of point sources (traffic vibrations) or extended machine foundations; moreover, the dynamic processes are not steady-state, harmonic but often shocks and transient vibrations.

In addition, the formulas presented result from regression analyses and therefore in any case involve some spread. Thus, we must query the usefulness of a model which provides A_r exactly to two places of decimals in the ideal case, if the influences mentioned above have to be taken into account by a roughly guessed correction factor. To people who are not familiar with the problem these models pretend a precision which does not exist.

HORIZONTAL COMPONENT

In the investigation by the plane strain BEM on the screening effect of infilled trenches to the horizontal vibration component, by Al-Hussaini/Ahmad (1991a), the wave field is generated by a strip foundation vibrating horizontally in the direction of the wave propagation. There are some differences in the results as compared to the vertical component isolation. In summary, smaller isolation effectiveness is obtainable. At the same value of S, the factor A_r is considerably higher than in the vertical component case. Moreover, the influence of the infill material stiffness is much smaller. The writer agrees with Liao (1992), who in the discussion points out that the compression wave length L_P should be an important parameter. If we bear in mind that at $\nu = 0.48$ the wave length ratio is $L_P/L_R \approx 5$, then at a distance of $5\ L_R$ between source and barrier the active isolation case should take place. It is surprising that obviously no influence of Poisson's ratio exists, at least within the considered range (ν = 0.3, 0.4, 0,48). Moreover, the observation that A_r increases with decreasing distance between source and infilled trench suggests the importance of other wave propagation processes than those considered up to now. As has been shown earlier, at a horizontally oscillating foundation this component of the body wave field is dominant over a relatively great distance, even at the surface (Fig. 9).

If the footing performs vertical vibrations, the horizontal component of the free wave field (far field) corresponds to the horizontal Rayleigh wave component. Chouw et al. (1991) have shown that in this case the screening effect of an open trench does not differ substantially from the case of the vertical Rayleigh wave component isolation. It seems that the screening of a wave field generated by a horizontally oscillating wave source cannot simply be explained by the wave propagation processes which have been proved to be responsible for the Rayleigh wave isolation.

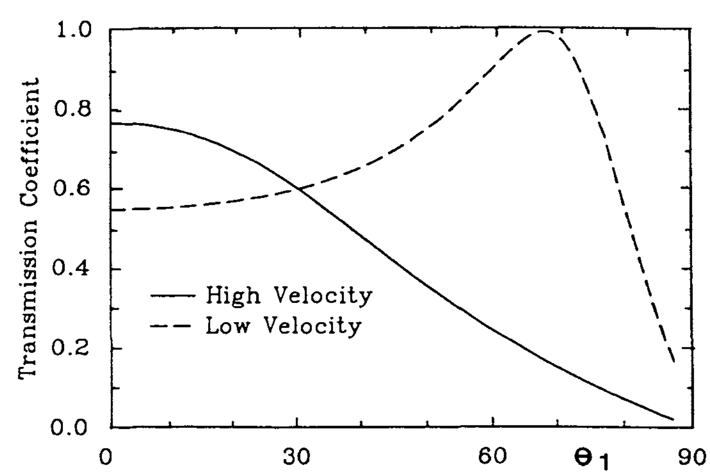


Fig. 33: Isolation effect of solid barrier depending on angle of incidence (Lee/Its, 1994)

INCREASE OF WAVE VELOCITY WITH DEPTH

The foregoing considerations were concerned with isolation measures in a homogeneous half-space, i.e. in a half-space with constant properties, especially elastic parameters, with depth. In reality, however, the normal case is that the wave velocity increases with depth. In addition, there is generally some layering or a ground water table. The usual assumption of layer interfaces parallel to the surface often represents yet an idealization. In the experimental investigations, this increase of wave velocity with depth is taken into account automatically. This must have some effect on the results, although in the model tests the variation of the Rayleigh wave velocity is small, due to the limited range of frequency. Woods (1968) and Dolling (1970) in pre-tests have found the wave velocity depending on depth by an approximate potential law, as should be expected, but with a non-zero value at the surface.

In their theoretical investigation, Ahmad et co. (1991) also considered the screening effect of open trenches in a half-space with a top layer of varying thickness. The ratio of shear wave velocities was in the range of 0.25 to 20. However, the geometric parameters are not normalized on the effective surface wave length in the two-layer system, but on the Rayleigh wave length of the upper layer material. Therefore, the results of systematic parameter variations are not applicable in practice. The same is true for part of the results from Leung et al. (1990), who performed plane strain BEM calculations on the screening effect of both open and infilled trenches, mainly in a two-layer system. From their results, the authors draw the conclusion that open trenches in a half-space with a soft material top layer must have a considerably larger depth (normalized on L_R of top layer) than in a homogeneous half-space, to yield the same isolation effectiveness. This tendency is pronounced in the case of concrete-filled trenches and at the horizontal surface wave component. In summary, the results are very unsystematic and no attempt is made to give a physical explanation.

The extended calculations using the plane strain BEM by Leung et al. (1991) deal with open trenches in a half-space with continuously increasing wave velocity. The soil models considered include the exponential law of shear modulus increase with depth from Vrettos (1990) and a linearly increasing modulus corresponding to an elastic Gibson's soil model. In this investigation, the normalization of the trench depth is based on the effective surface wave length L'_R. With both models of the inhomogeneous half-space, considerably larger normalized trench depths are required to obtain satisfactory shielding effect, as may be seen from Fig. 34. Thus, by considering realistic soil models, the theoretical investigation results confirm the experimental findings quite well.

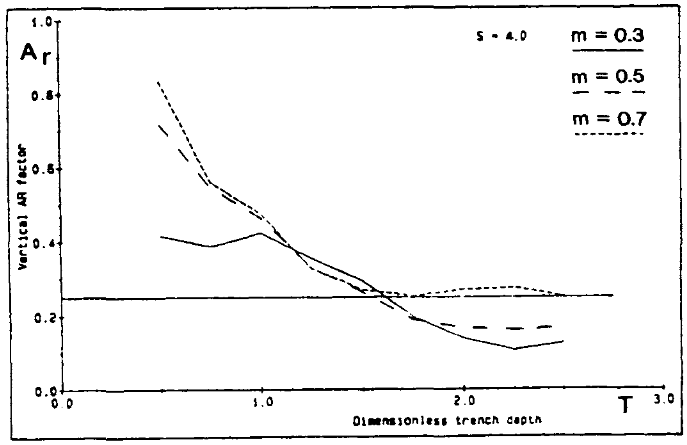


Fig. 34: A_r dependent on T for Gibson's soil model, open trench (Leung et al., 1991)

4.5 ROWS OF BORE-HOLES OR PILES

Only relatively few investigation reports on this subject have been published, though this kind of barrier is somewhat attractive, because it seems to be performed much more easily in reality. The reasons are the difficulties in test performance on the one hand, and in treating a true three-dimensional, complicated problem theoretically on the other. No numerical solutions are available. Moreover, the number of important parameters is increased. At a row of piles, all those parameters which are important at a solid barrier have to be considered, too, but instead of the width of the barrier, in this case the diameter and the spacing of the piles are significant. The casing of bore holes might also be of influence.

EXPERIMENTAL INVESTIGATIONS

The measurements by Woods et al. (1974) were carried out by holographic interferometry at rows of bore-holes in sand on an extremely reduced scale. As a result, a systematic relationship between the geometrical parameters and the screening effectiveness was established . To eliminate the normalized depth as a variable in the tests, a value of T ≈ 1.4 was used. For the same reason the length of the rows of holes was chosen to be about 2.5 L_R.

A typical picture of the tests is presented in Fig. 35: The narrow lines around the vibrating foundation indicate high amplitudes, whereas behind the barrier the amplitude level is very low. In the diagram in Fig. 36, which shows the results of the systematic test series, Woods et al. used the "effectiveness" as a measure of the isolation quality of a barrier instead of the usually applied amplitude reduction factor. As did all other researchers, they found that A_r decreases, as

- the diameter of the holes increases,
- the clear spacing of the holes decreases.

The shaded zone in Fig. 36 is well established, whereas the dashed lines are speculative, because they are based on only a few tests. Thus, a row of uncased bore holes can be expected to yield a satisfactory isolation effectiveness ($A_r \leq 0.25$), if the normalized diameter is $D/L_R = 0.17$ and the spacing between the holes about 0.1 L_R. It was observed by the authors that an increase of the diameter beyond about $^1/_6$ L_R does not improve screening efficiency.

In the model tests on rows of bore holes by Haupt (1981) the amplitude reduction factor A_r was related to the normalized shielding area $T \cdot D/(D + E)$, where T is the depth as defined in Fig. 15, D is the diameter and E the clear distance between the holes. In these experiments the value of A_r was found to be much higher than in the tests described above, i.e. in the range of 0.6

Fig. 35: Isolation effect of rows of bore holes, holographic interferometrie (Woods, et al., 1974)

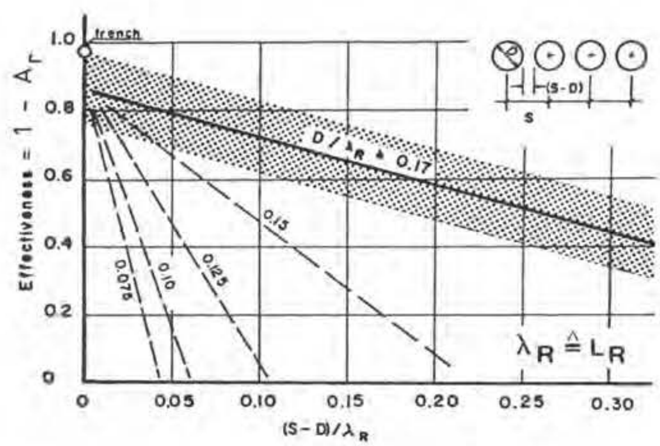


Fig. 36: Effectiveness of rows of open holes dependent on geometric parameters (Woods et al., 1974)

to 0.9. One reason might be the smaller depth of the holes ranging between T = 0.4 and 1.3 at maximum. The main reason, however, for the unsatisfying effectiveness of the barriers is to be seen in the casing of the holes, perfomed by soft plastic tubes (polyethylene). On this reduced scale even soft material, thin-wall tubes behave like rigid casings, transferring energy across the barrier. However, this is the situation encountered in reality, where a bore hole of half a meter or more in diameter must be stabilized by steel tubes, unless it is driven into rock. The above mentioned relation was verified by Haupt in an experiment, where after the accomplishment of the test the tubes were removed and the measurements repeated. The amplitude reduction factor was improved from $A_r = 0.76$ (with tubes) to $A_r = 0.5$ (without tubes), the latter value corresponding fairly well with the results from Woods et al. Thus, is should be kept in mind that casing the bore-hole might considerably impair the isolation effectiveness of the rows.

Xian-Jian (1991) has reported on two cases of vibration isolation by holes and piles. In the first case, with a double row of bore holes (diameter = 0.4 m, clear spacing = 0.5 m, depth = 8 m) in a soil composed of loess and clay (L_R = 3.85 m), he observed an amplitude reduction down to about 20 % within a confined area behind the rows. In the second case, with the same soil (V_R = 154 m/s) the floor inside a building was screened from vibrations by the foundation piles (diameter = 0.8 m), arranged as a row at the periphery of the building. To improve the effect, a row of holes was bored around the building. Unfortunately, the author does not describe technical details. The isolation effectiveness was fairly good (A_{rd} = 0.25 - 0.52), but not as good as had been prognosticated. The author uses a relatively simple relation to determine the amplitude reduction factor of rows of bore holes or piles, also as double rows. It is based on the effect of difference in wave impedance of the barrier and the surrounding soil. The wave impedance of the rows of holes depends on their normalized diameter and spacing. At piles the material properties are also taken into account.

The screening effect of rows of piles on propagating P-wave pulses in water are investigated experimentally on a model scale by Liao/Sangrey (1978). Although, due to the special kind of waves, the results cannot be used as a basis for a rational design of wave barriers in soil, some tendencies have become quite clear: again a better screening effect was observed at increasing diameter and decreasing spacing of the piles. Soft material, which was used to simulate open holes, showed a considerably better effect than stiff pile material. Similar to the observations by Woods et al. (1974) on open holes, the authors found that greater pile diameters than about 0.17 L_R do not increase the isolation effectiveness. It was also observed that at relatively large spacing of the piles a second row may increase screening efficiency, but reducing the spacing of the first row is at least as effective.

THEORETICAL INVESTIGATIONS

For the purpose of practical design of vibration barriers, the theoretical investigations by Avilés/Sánchez-Sesma (1983, 1988) and by Boroomend/Kaynia (1991) can be used only with restrictions. In the three papers, rows of piles are considered with the pile material having a 1,000 times higher Young's modulus than the surrounding material. In practice this is unrealistic. With the ratio of the moduli being only 100, which is rather a realistic order of magnitude, the isolation effectiveness decreased strongly (Boroomand/Kynia). The analytical solutions are all based on the consideration of diffraction and scattering of the waves at the piles. Partly, infinitely long, rigid piles in the full-space (P-, SV- and SH-waves) are treated (Avilés/Sánchez-Sesma, 1983). The limiting case, that the row of piles becomes a solid barrier of large normalized depth, if the clear spacing between the piles tends to zero, cannot be considered.

All investigations are in agreement with the aforementioned results concerning the influence of diameter and spacing. However, in the calculations by Avilés/Sánchez-Sesma (1988) a satisfactory screening effect ($A_r < 0.2$) in the case of isolating Rayleigh waves is obtained only at a diameter of 0.4 L_R, clear spacing of 0.3 L_R and a pile length of $2L_R$ ($E_B/E = 1000$). In this case, the amplitude reduction factor is not an average value like A_r or A_{rd} as defined in previous chapters, but a number valid for a confined area at a distance 20 to 40 L_R behind the barrier. Thus, the theoretical investigations briefly outlined above yield little practical information for the design of vibration barriers in soil.

4.6 OTHER ISOLATION MEASURES

Chouw/Schmid (1991) have proposed a new method of vibration isolation, in which a large, stiff concrete slab is placed in the soil below a foundation. The effect is based on the fact that waves cannot propagate in a layer on top of a rigid half-space, if the thickness of the layer is smaller than the critical thickness, which in the case of Rayleigh waves can be taken as $L_S/4$. Fig. 37 depicts the function of the vertical vibration component dependent on the distance from the source, as has been obtained from a plane strain BEM calculation. If the concrete slab is located closely below the foundation, the isolation effectiveness is considerable. The authors have shown that this method can effectively be applied as an active isolation measure, too. Tests on an almost natural scale have been carried out recently by Forchap et al. (1994). The concrete slab, 5.0 m x 5.0 m in size and 0.6 m thick, was situated 1.0 m below the surface, where the foundation block (1.0 m x 1.0 m x 0.5 m) was located. The shear wave velocity showed values between 250 m/s and 160 m/s. In Fig. 38, the results are presented as amplitudes of the horizontal and vertical displacement components at the surface, depending on frequency. From the change of the resonance curve it is to be concluded

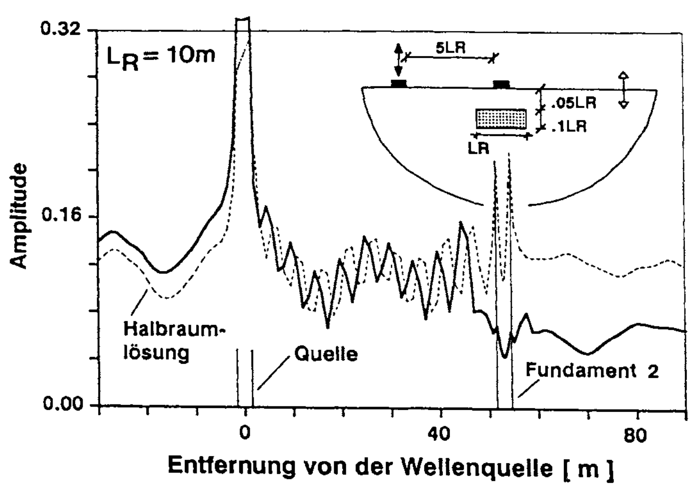


Fig. 37: Vibration isolation by a concrete slab below the foundation (Chouw/Schmid, 1991)

that the vibrating system has changed, certainly from a one-mass to a two-mass system. To what extent a convincing isolation measure is obtained by this system, apart from the change in resonance behaviour, is not yet clearly recognizable. Gaul/Plenge (1993) observed poor active isolation efficiency in their holographic interferometry tests.

Another way of reducing the vibration amplitudes from highway traffic, during progagation into the vicinity, was proposed by Taniguchi/Okada (1981). They reported on measurements by which they proved a successful reduction of the vibration level down to about 20 to 50 % due to the improvement of the soil. These good results, however, were obtained only at low frequencies (< 10 Hz). The improvement of the soft clay was performed by lime piles of 12 m depth. FEM calculations carried out by the authors have confirmed that the vibration amplitude reduction was caused by the change in dynamic soil properties.

Surface waves in urban areas are permanently reflected and scattered by the foundations of buildings and are consequently diminished in amplitude. This phenomenon is frequently observed, but apparently it has not been used intentionally for the design of isolation measures, probably because of a lack of reliable knowledge. In a theoretical investigation, Kovacs (1987) has considered the interaction between a steady-state harmonic wave field and an array of building foundations. He developed a method of calculating the amplitude reduction, taking into account the individual position and size of each foundation bloc.

4.6 PRACTICAL CONSIDERATIONS

Present research work on the isolation of vibrations by open and infilled trenches or other barriers in the half-space, has yielded plenty of important knowledge on the behaviour of elastic waves in the presence of an inhomogeneity. We have, however, to face the question of their practical significance. For the following considerations let us assume that the amplitude reduction factor is required to be $A_r \leq 0.25$.

In reality, in many cases the prevailing Rayleigh wave length will typically be in the order of 8 m (V_R = 200 m/s; f = 25 Hz) at the minimum possibly 3 m (V_R = 120 m/s; f = 40 Hz). Taking into account the embedment of a machine foundation of about 1.2 m, an open trench as active isolation measure must have a

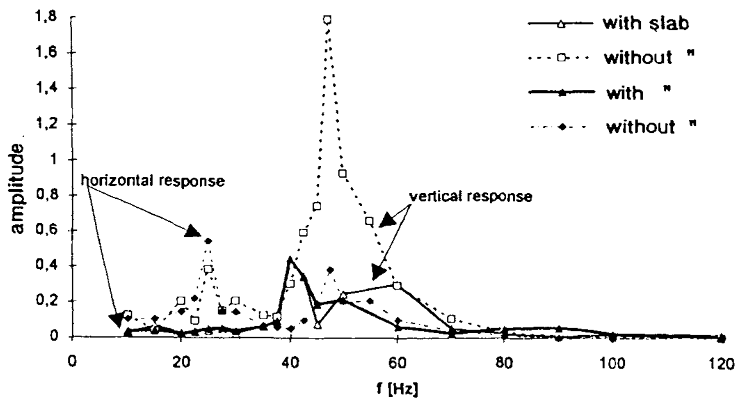


Fig. 38: Experimental results on isolation by concrete slab (Forchap et al., 1994)

depth of (0.6 · 8 + 1.2) = 6 m and as passive isolation a depth in the order of (1.3 · 8) = 10.4 m. Trenches of this depth cannot be kept open without special and expensive stabilizing structures. Bentonite suspension as infill material requires permanent maintenance. Even open trenches with depths of 3.0 m (active) or 3.9 m (passive case), which are necessary at the minimum wave length, are unlike to be produced. In the active case, one would prefer isolation of the machine by spring elements or rubber pads.

Concrete barriers in the far field would need a cross-sectional area of about 1.1 · L^2_R, which in the most favorable case (L_R = 3 m) means a value of about 10 m². Hence, one must build e.g. a concrete core wall of 1 m thickness down to a depth of 10 m. In the usual case (L_R = 8 m) the wall theoretically should have a thickness of 4 m and a depth of 17.6 m. The unfavorable influence of increasing wave velocity with depth has not yet been taken into account. Also the method with the concrete slab below the foundation, proposed by Chouw, though theoretically seeming advantageous, is not realistic from the technical standpoint. Rows of bore holes would require hole diameters of 1.4 m with spacing of about 0.8 m and a minimum depth of 12 m. At concrete piles these dimensions would be at least in the same order of magnitude. Only with the minimum value of L_R the dimensions of rows of bore holes are in a moderate range (diameter ≈ 0.5 m, spacing ≈ 0.3 m). Summarizing, with regard to the difficulties mentioned and the costs to be expected, it is more than likely that isolation measures of the kind considered above will be applied only very seldom, probably only if all other possibilities fail, unless the requirements to their effectiveness are essentially lowered.

One expedient seems to appear with a new technique of keeping deep trenches open, which is reported by Massarch (1991). He describes the so-called gas cashion screen. It is built up of horizontally placed, gas-inflated tubes, which are overlapping in the vertical direction and are enveloped by a woven geotextile fabric, see Fig. 39. The cushions are manufactured of a thin-walled flexible plastic laminate, which consists of two layers of polyethylene film, surrounding a thin aluminium foil. The cushion panels are lowered into a trench, filled with cement/bentonite grout, using a heavy concrete weight attached to the lower end of the screen to avoid buoyancy. After hardening of the slurry, an open trench is obtained stabilized by the gas tubes under pressure.

Massarsch (1991) describes several systematic investigations to prove the effectiveness of the gas cushion panels as a vibration isolation barrier. Measurements were performed on a screen 20 m long and 12 m deep. The surface wave length was in the range of 4 m to 25 m. With steady-state harmonic vibrations, an amplitude reduction factor of $A_r \approx 0.2 - 0.25$ was obtained. Further tests on the screening effectiveness of a panel of 6 m depth with respect to shocks, generated by explosive charges detonating at a distance of 6 m, showed quite good performance ($A_r \approx 0.2 - 0.35$). At the first application of this technique in Germany, reported by Schiffer (1992), two residential buildings were protected against railroad vibrations by such a trench 8 m deep and 40 m long. At the measuring points, located at the ground surface behind the trench and within one of the buildings, the reduction factor of the maximum vibration values was in the range of 0.4 to 0.6.

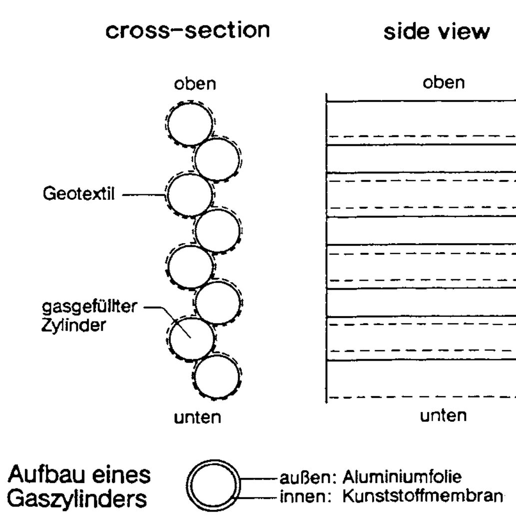


Fig. 39: Gas cushion panel as isolation measure (Schiffer, 1992)

The essential problem, however, is the stability of the gas cushions. If some of the tubes are damaged during placing of the panel, a strip of direct contact of both trench sides results, presumably reducing the isolation effect considerably. Also, if it cannot be avoided that the tubes lose pressure with time, the trench could collapse due to active earth pressure and become useless. In addition, nearby structures could become unstable. Experience from Sweden shows that all trenches of this kind, as far as examined, remained intact over a period of five years. On the other hand, Massarsch (1991) has reported on a panel which has lost markedly in effectiveness due to surface water penetrating between the cushions and the ground layer.

5 CLOSURE

The present report, which deals with the propagation of elastic waves in the soil under various conditions, has tried to present on the one hand the basic relationships necessary to all engineers in contact with dynamic problems, and on the other hand to include recent research results. For the isolation measures an overview of most of the important investigations and experiences is given. There is a relatively large amount of theoretical research work. With the FEM and BEM developed in recent decades complicated systems can also be treated. Further theoretical investigations concerning isolation measures should include the interior of the half-space in order to clear up the way in which a wave field is affected by any kind of inhomogeneity. The wide, shallow open trenches as well as the subsurface barriers are of special interest. As long as the amplitude is considered only at the surface of the half-space, all attempts to explain the physical phenomena causing the isolation effect remain somewhat speculative.

There is only a relatively small number of experimental results. Tests are difficult to carry out and require skilled personnel. Due to the reasons outlined in the foregoing chapter, isolation measures are carried out seldom and therefore, only few measurements and practical experience with barriers in natural scale are available. Experimental investigations should be directed to the development of measures which are practical in application, stable in the long term, and inexpensive. The gas cushion panel is a first step. Measurements on rows of bore holes on the natural scale have not yet been published. On the other hand, due to the development of the SASW-method, the propagation of waves in a layered half-space has more often been the subject of experimental studies, which has helped to improve understanding of the wave propagation in real soil.

REFERENCES

Aboudi, J. (1973), "Elastic Waves in Half-Space with Thin Barrier", Proc. ASCE, No. EM1, Febr.

Achenbach, J.D. (1976), "Wave Propagation in Elastic Solids", Applied Mathematics and Mechanics, North-Holland/American Elsevier, 1973/1976.

Ahmad, S., T.M. Al-Hussaini (1991), "Simplified Design for Vibration Screening by Open and In-filled Trenches", Proc. ASCE Vol. 117, No. GT1, Jan.

Ahmad, S., T.M. Al-Hussaini, K.L. Fishman (1994), "An Investigation on Active Isolation of Machine Foundations by Open Trenches", Proc. ASCE, Journ. of Geotechn. Engg, accepted for publication.

Al-Hussaini, T.M., S. Ahmad (1991a), "Design of Wave Barriers for Reduction of Horizontal Ground Vibration", Proc. ASCE, Vol. 117, No. GT4, April.

Al-Hussaini, T.M., S. Ahmad (1991b), "Simple Design Methods for Vibration Isolation by Wave Barriers", Proc. 2nd Int. Conf. on Rec. Adv. in Geotech. Earthqu. Engg and Soil Dyn., St. Louis, Missouri.

Allen, N.F., F.E. Richart, R.D. Woods (1980), "Fluid Wave Propagation in Saturated and Nearly Saturated Sands", Proc. ASCE, Vol. 106, No. GT3, March.

Avilés, J., F.J. Sanchéz-Sesma (1983), "Piles as Barriers for Elastic Waves", Proc. ASCE, Vol. 109, No. GT9, Sept.

Avilés, J., F.J. Sanchéz-Sesma (1988), "Foundation Isolation from Vibrations Using Piles as Barriers", Proc. ASCE, Vol. 114, No. GT11, Nov.

Banerjee, P.K., S. Ahmad, K. Chen (1989), "Advanced Application of BEM to Wave Barriers in Multi-Layered Three-Dimensional Soil Media", Earthqu. Engg and Struct. Dyn., Vol. 16, 1041 - 1060.

Bardet, J.P., H. Sayed (1993), "Velocity and attenuation of compressional waves in nearly saturated soils", Soil Dyn. and Earthqu. Engg, Vol. 12 (1993), 391 - 401.

Barkan, D.D. (1962), "Dynamics of Bases and Foundations", McGraw-Hill Book Co., New York.

Beskos, D.E., B. Dasgupta, I.G. Vardoulakis (1986), "Vibration isolation using open or filled trenches; Part 1: 2-D homogeneous soil", Comput. Mech. 1986, No. 1, Springer.

Biot, M.A. (1956), "Theory of Propagation of Elastic Waves in a Fluid-Saturated Porous Solid", J. of Acoust. Soc. of America, Vol. 28, March, 168 - 191.

Bornitz, G. (1931), "Über die Ausbreitung der von Großkolbenmaschinen erzeugten Bodenschwingungen in die Tiefe", Verlag Julius Springer, Berlin 1931.

Boroomand, B., A.M. Kaynia (1991), "Vibration Isolation by an Array of Piles", Proc. 5th. Int. Conf. on Soil Dyn. and Earthqu. Engg (SDEE '91), Karlsruhe (Germany), Sept. 1991.

Caloi, P. (1948), "Comportement des Ondes de Rayleigh dans un Milieu Firmo-Elastique Indefini", Publ. Bur. Central Seism. Int., A, 17, 1948.

Chouw, N., R. Le, G. Schmid (1991), "Vibration transmitting behavior of the soil", Proc. of the Europ. Conf. on Struct. Dyn. (Eurodyn '90), Univ. of. Bochum, Germany, A.A. Balkema.

Chouw, R.L., G. Schmid (1991), "Verfahren zur Reduzierung von Fundamentschwingungen und Bodenerschütterungen mit dynamischem Übertragungsverhalten einer Bodenschicht", Bauingenieur, Vol. 66, 215 - 221.

Ciesielski, R., A. Zieba (1991), "Experimental investigation on reduction of impact vibrations through ground by means of shallow vertical trenches", Proc. of the Europ. Conf. on Struct. Dyn. (Eurodyn '90), Univ. of Bochum, Germany, A.A. Balkema.

Cramer, H., W. Wunderlich (1991), "Multiphase models in soil dynamics", Proc. of the Europ. Conf. on Struct. Dyn. (Eurodyn '90), Univ. of Bochum, Germany, A.A. Balkema.

Dasgupta, D., D.E. Beskos, I.G. Vardoulakis (1990), "Vibration isolation using open or filled trenches; Part 2: 3-D homogeneous soil", Computational Mech. 1990, No. 6, Springer.

DeBarros, F.C.P., J.E. Luco (1993), "Diffraction of obliquely incident waves by a cylindrical cavity embedded in a layered viscoelastic half-space", Soil Dyn. and Earthqu. Engg, Vol. 12, 159 - 171.

DeBremaecker, J.C. (1958), "Transmission and Reflection of Rayleigh-Waves at Corners", Geophysics, Vol. XXIII, No. 2, April.

Diankui, L., H. Feng (1991), "Scattering of plane SH-wave by cylindrical canyon of arbitrary shape", Soil Dyn. and Earthqu. Engg, Vol. 10, No. 5, July.

Dolling, H.J. (1965), "Schwingungsisolierung von Bauwerken durch tiefe, auf geeignete Weise stabilisierte Schlitze", VDI-Bericht Nr. 88, 1965.

Dolling, H.J. (1970), "Abschirmung von Erschütterungen durch Bodenschlitze", Die Bautechnik No. 5 and 6, 1970.

Dresen, L. (1972), "Modellseismische Untersuchungen zum Problem der Ortung oberflächennaher, kreiszylindrischer Hohlräume in Festgestein", Bericht des Inst. f. Geophysik, Univ. Bochum (Germany), No. 1, May.

EAD (1992), "Empfehlungen des Arbeitskreises Nr. 9, Baugrund dynamik", German Geotechnical Society, Die Bautechnik, Vol. 69 (1992), Heft 9, 518 - 534.

Emad, K., G.D. Manolis (1985), "Shallow Trenches and Propagation of Surface Waves", Proc. ASCE, Vol. 111, No. EM2, Febr.

Erlingsson, S., A. Bodare (1992), "Vertical S waves in soil stratum over halfspace", Soil Dyn. and Eartqu. Engg, Vol. 11 (1992), 427 - 434.

von Estorff, O., A.L. Pais, E. Kausel (1990), "Some Observations on Time Domain and Frequency Domain Boundary Elements", Int. Journ. for Num. Meth. in Engg, Vol. 29, 785 - 800.

von Estorff, O., H. Antes (1991), "Numerische Studien zur Schwingungsisolierung durch Schlitze", Proc. of the D-A-CH Confer. 1991, Zürich, Oct.

Ewing, W.M., W.S. Jardetzky, F. Press (1957), "Elastic Waves in Layerd Media", McGraw-Hill, New York.

Fei, H.-C., F.E. Richart (1991), "Effect of Triaxial Stresses von Shear Wave Propagation", Proc. 5th Int. Conf. on Soil Dyn. and Earthqu. Engg (SDEE '91), Karlsruhe (Germany), Sept.

Forchap, E., Th. Siemer, G. Schmid, H.L. Jessberger, W. Rücker (1994), "Experiments to investigate the reduction of soil wave amplitudes using a built in block", Proc. Int. Conf. on Earthqu. Resist. Constr. and Design at Berlin 1994, Balkema, Rotterdam.

Gangi, A.F. (1967), "Experimental Determination of P-Wave/Rayleigh Wave Conversion Coefficients at the Apex of a Stress-Free Wedge", Journ. of Geophys. Res., Vol. 72, No. 22.

Gaul, L., M. Plenge (1992), "Progress in 3D-BE Calculations and Optoelectronic Measurements of Soil-Structure Interaction", Advanced Dyn. Anal. by BEM, Devel. in BEM-7 (ed. by Banerjee/Kobayashi), Elsevier, London.

Gaul, L., M. Plenge (1993), "Baugrundeinflüsse auf das Schwingungsverhalten von Fundamenten", VDI-Berichte Nr. 1082, VDI-Verlag 1993, 195 - 220.

Gazetas, G. (1982), "Vibrational Characteristics of Soil Deposits with Variable Wave Velocity", Int. Journ. for Num. and Analyt. Meth. in Geomechanics, Vol. 6, 1982, 1 - 20.

Gazetas, G. (1991), "Foundation Vibration" in: Foundation Engineering Handbook (2. ed.), edited by H.-S. Fang, van Nostrand Reinhold, New York.

Gibson, R.E. (1967), "Some results concerning displacement and stresses in an non-homogeneous elastic half-space", Geotechnique 1967, Vol. 17, 58 - 67.

Gilbert, F., L. Knopoff (1960), "Seismic Scattering from Topographic Irregularities", Journ. of Geophys. Res., Vol. 65, No. 10, Oct.

Gucunski, N. (1991), "Generation of Low Frequency Rayleigh Waves for the Spectral-Analysis-of-Surface-Waves Method", Ph. D. Dissertation, Univ. of Michigan, Ann Arbor, Mich.

Gucunski, N., R.D. Woods (1991), "Use of Rayleigh Modes in Interpretation of SASW Test", Proc. 2nd Int. Conf. on Rec. Adv. in Geotech. Earthqu. Engg and Soil Dyn., St. Louis, Missouri.

Hardin, B.O. (1961), "Study of Elastic Wave Propagation and Damping in Granular Materials", Ph. D. Dissertation, Univ. of Florida, Aug.

Hardin, B.O., W.L. Black (1966), "Sand Stiffness under Various Triaxial Stresses", Proc. ASCE, Vol. 92, No. SM2, March.

Haupt, W.A. (1977), "Isolation of Vibrations by Concrete Core Walls", Proc. 9. Int. Conf. on Soil Mech. and Found. Engg, Tokyo, July.

Haupt, W.A. (1978), "Surface-waves in non-homogeneous half-space", Proc. Dyn. Meth. in Soil and Rock Mech. (DMSR 77), Vol. I, A.A. Balkema, Rotterdam.

Haupt, W.A. (1978a), "Verhalten von Oberflächenwellen im inhomogenen Halbraum mit besonderer Berücksichtigung der Wellenabschirmung", Dr.-Ing. Thesis, Report of the Inst. of Soil and Rock Mech., Univ. of Karlsruhe, Germany, No. 74.

Haupt, W.A. (1978b), "Numerical methods for the computation of steady-state harmonic wave fields", Proc. Dyn. Meth. in Soil and Rock Mech. (DMSR 77), Vol. I, A.A. Balkema, Rotterdam.

Haupt, W.A. (1981), "Model Tests on Screening of Surface Waves", Proc. X Int. Conf. on Soil Mech. and Found. Engg, Stockholm.

Haupt, W.A. (1985), "Erschütterungsabschirmung in gefrorenem Boden (Vibration isolation in frozen soils)", Public. of the Inst. for Soil Mech. of the Landesgewerbeanstalt, Heft 43, Nürnberg (Germany), 1985.

Haupt, W.A. (1986), "Ausbreitung von Wellen im Boden", in: Bodendynamik - Grundlagen und Anwendung", Vieweg-Verlag Braunschweig/Wiesbaden (Germany).

Heisey, J.S., K.H. Stokoe II, A.H. Meyer (1982), "Moduli of Pavement Systems from Spectral Analysis of Surface Waves", Transportation Research Record, No. 852, Washington, D.C., Jan.

Hiltunen, D.R., R.D. Woods (1988), "SASW and Crosshole Test Results Compared", Earthqu. Engg and Soil Dyn. II, ASCE Geotechn. Spec. Publ. No. 20.

Hiltunen, D.R., N. Gucunski (1994), "Annotated Bibliography on SASW", Geophysical Characterization of Sites, ISSMFE Techn. Com. No. 10 (R.D. Woods), Oxford & IBM Publishing Co., New Delhi.

Hirai, H. (1992), "Analysis of Rayleigh waves in saturated porous elastic media by finite element method", Soil Dyn. and Earthquake Engg, Vol. 11 (1992), 311- 326.

Hudson, J.A., L. Knopoff (1964), "Transmission and Reflection of Surface Waves at a Corner; 2. Rayleigh-Wave", Journ. of Geophys. Res., Vol. 69, No. 2, Jan.

Hudson, J.A. (1967), "Scattered Surface Waves from a Surface Obstacle", Geophys. Journ. Royal Astr. Soc., No. 13.

Ishihara, K. (1970), "Approximate Forms of Wave Equations for Water Saturated Porous Materials and Related Dynamic Modulus", Soils and Foundations, Vol. 10, No. 4, 1970.

Kane, J., J. Spence (1963), "Rayleigh-Wave Transmission on Elastic Wedges", Geophysics, Vol. XXVIII, No. 5, Part I, Oct.

Knopoff, L. (1959a), "Scattering of Compression Waves by Spherical Obstacles", Geophysics, Vol. XXIV, No. 1, Feb.

Knopoff, L. (1959b), "Scattering of Shear Waves by Spherical Obstacles", Geophysics, Vol. XXIV, No. 2, April.

Kolsky, H. (1963), "Stress Waves in Solids", Dover Publ. Inc., New York 1963.

Kovács, I. (1987), "Zum Entwurf der Gründungen schwingungsempfindlicher Produktionsstätten", Die Bautechnik, Heft 5, 1987, 145 - 158.

Lapwood, E.R. (1961), "The Transmission of a Rayleigh Pulse around a Corner", Geophysics, Vol. 26, No. 4.

Lee, J.S., Its, E.N. (1994), "Surface Waves of Oblique Incidence Across Deep In-Filled Trenches", Proc. ASCE, Journ. of Engg Mechanics, fall 1994.

Lee, N.J. (1991), "Inherent and Stress-Induced Anisotropy of Sand Determined by Body Wave Velocities", Ph.D. Dissertation, University of Texas, Austin.

LeHouédec, D., S. Malek, J. Picard (1991), "Effectiveness of trenches or screens for scattering surface waves", Proc. of the Europ. Conf. on Struct. Dyn. (Eurodyn '90), Univ. of Bochum, Germany, A.A. Balkema.

Leung, K.L., D.E. Beskos, I.G. Vardoulakis (1990), "Vibration isolation using open or filled trenches; Part 3: 2-D non-homogeneous soil", Comp. Mech. 1990, No. 7, 137 - 148.

Leung, K.L., I.G. Vardoulakis, D.E. Beskos, J.L. Tassoulas (1991), "Vibration Isolation by Trenches in Continuously Non-homogeneous Soil by the BEM", Soil Dyn. and Earthqu. Engg, Vol. 10, No. 3, April.

Liao, S., D.A. Sangrey (1978), "Use of piles as Isolation Barriers", Proc. ASCE, Vol. 104, No. GT9, Sept.

Liao, S.S.C. (1992), "Discussion to: Design of Wave Barriers for Reduction of Horizontal Ground Vibration", Proc. ASCE. Vol. 118, No. GT8, Aug.

Love, A.E.H. (1944), "A Treatise on the Mathematical Theory of Elasticity", Dover Publ. Inc., New York, forth Ed. 1944.

Lysmer, J., G. Waas (1972), "Shear Waves in Plane Infinite Structures", Proc. ASCE, No. EM1, Febr.

Massarsch, K.R. (1991), "Ground Vibration Isolation Using Gas Cushions", Proc. 2nd Int. Conf. on Rec. Adv. in Geotech. Earthqu. Engg and Soil Dyn., St. Louis, Missouri.

May, T.W., B.A. Bolt (1982), "The Effectiveness of Trenches in Reducing Seismic Motion", Earthqu. Engg and Struct. Dyn., Vol. 10, 195 - 210.

McCamy, K., R.P. Meyer, T.J. Smith (1962), "Generally Applicable Solutions of Zoeppritz' Amplitude Equations", Bull. of the Seism. Soc. of America, Vol. 52, No. 4, Oct.

McNeill, R.L., B.E. Margason, F.M. Babcock (1965), "The Role of Soil Dynamics in the design of stable test pads", Proc. Guidance and Control Conf., Minneapolis, Minnesota.

Medearis, K. (1979), "Dynamic Characteristics of Ground Motions Due to Blasting", Bull. of the Seism. Soc. of America, Vol. 69, No. 2, 627 - 639.

Miller, G.F., H. Pursey (1955), "On the Partition of Energy Between Elastic Waves in a Semi-Infinite Solid", Proc. Royal Society, London, Series A, Vol. 233, 55 - 69.

Neumeuer, M. (1963), "Untersuchungen über die Abschirmung eines bestehenden Gebäudes gegen Erschütterungen beim Bau und Betrieb einer U-Bahnstrecke", Baumaschinen und Bautechnik, Jahrg. 10, Heft 1, Jan.

Nogami, T., M. Kazama (1992), "Dynamic response analysis of submerged soil by thin layer element method", Soil. Dyn. and Earthqu. Engg, Vol. 11 (1992), 17 - 26.

Pilant, W.L., L. Knopoff, F. Schwab (1964), "Transmission and Reflection of Surface Waves at a Corner; 3. Rayleigh-Waves, Exp.", Journ. of Geophys. Res., Vol. 69, No. 2, Jan.

Plenge, M. (1991), "Erfassung der Erschütterungsausbreitung im Boden mit Hilfe optoelektronischer Messverfahren", Erdbebeningenieurwesen, Publ. No. 5, Germ. Soc. of Soil Dyn. and Earthqu. Engg, Potsdam, Germany.

Prakasch, S. (1981) "Soil Dynamics", McGraw-Hill Inc., New York

Prange, B., W.A. Haupt (1974), "Anomalien eines stationären Wellenfeldes an der Halbraumoberfläche infolge von Untergrundeinschlüssen", Report to the Deutsche Forschungsgemeinschaft No. Pr 102/2, May 1974.

Rayleigh, Lord J.W.S. (1885), "On Waves Propagated along the Plane Surface of an Elastic Solid", London Math. Soc., Proc. 17, 1885.

Richart, F.E., J.R. Hall, R.D. Woods (1970), "Vibrations of Soils and Foundations", Prentice-Hall, Englewood Cliffs, N.J.

Rix, G.J. (1988), "Experimental Study of Factors Affecting the Spectral-Analysis-of-Surface-Waves Methods", Ph.D. Dissertation, Univ. of Texas, Austin, August.

Roesler, St.K, (1979), "Anisotropic Shear Modulus due to Stress Anisotropy", Proc. ASCE, Vol. 105, No. GT7, July.

Roësset, J.M., D.-W. Chang, K.H. Stokoe II (1991), "Comparison of 2-D and 3-D Models for Analysis of Surface Wave Tests", Proc. 5th Int. Conf. on Soil Dyn. and Earthqu. Engg (SDEE '91), Karlsruhe (Germany), Sept. 1991.

Schiffer, W. (1992), "Schlitzwandbauweise mit Gasmattenelementen zur Abschirmung von Erschütterungen", Proc. of the STUVA-Symp. at Düsseldorf 1991, STUVA, Köln, Germany.

Segol, G., P.C.Y. Lee, J.F. Abel (1978), "Amplitude Reduction of Surface Waves by Trenches", Proc. ASCE, No. EM3, June.

Stokoe II, K.H., S.H.H. Lee, D.P. Knox (1985), "Shear Moduli Measurements Under True Triaxial Stresses", Advances in the Art of Test. Soils under Cyclic Cond., Published by the ASCE (Khosta), Oct. 1985.

Stokoe II, K.H., S. Nazarian (1985), "Use of Rayleigh Waves in Liquefaction Studies", Measurement and Use of Shear Wave Velocity for Evaluating Dyn. Soil Properties. (Ed. Woods), Publ. by ASCE, May 1985.

Stokoe II, K.H., J.N. Lee, S.H. Lee (1991), "Characterization of Soil in Calibration Chambers with Seismic Waves", Proc. 1st Int. Symp. on Calibration Chamber Testing, Potsdam, New York, June.

Stokoe II, K.H., S.G. Wright, J.A. Bay, J.M. Roësset (1994), "Characterization of Geotechnical Sites by SASW Method", Geophysical Characterization of Sites, ISSMFE Techn. Com. No. 10 (Ed. Woods), Oxford & IBH Publishing Co., New Delhi.

Stonely, R. (1924), "Elastic Waves at the Surface of Separation of Two Solids", Proc. Roy. Soc. (London), Series A, Vol. 106.

Studer, J., A. Ziegler (1986) "Bodendynamik", Springer-Hochschultext, Berlin/Heidelberg.

Taniguchi, E., S. Okada (1981), "Reduction of Ground vibrations by Improving Soft Ground", Soils and Foundations, Vol. 21, No. 2, June.

Todorovska, M.I., V.W. Lee (1991), "Surface motion of shallow circular alluvial valleys for incident plane SH waves-analytical solution", Soil Dyn. and Earthqu. Engg, Vol. 10, No. 4, May.

Tokimatsu, K., S. Tamura, H. Kojima (1992a), "Effects of Multiple Modes on Rayleigh Wave Dispersion Characteristics", Proc. ASCE, Vol. 118, No. GT 10, Oct.

Tokimatsu, K., K. Shinzawa, S. Kuwayama (1992b), "Use of Short-Period Microtremors for V_S Profiling", Proc. ASCE, Vol. 118, No. GT10, Oct.

Vardoulakis, I.G., D.E. Beskos (1986), "Dynamic Behavior of nearly saturated porous media", Mech. of Materials, Vol. 5, 87 - 108.

Vrettos, Ch. (1990), "In-Plane Vibrations of Soil Deposits with Variable Shear Modulus: I. Surface Waves", Int. Journ. for Num. and Analyt. Meth. in Geomechanics, Vol. 14, 1990, 209 - 222

Vrettos, Ch. (1991), "Time-Harmonic Boussinesq Problem for a Continuously Non-Homogeneous Soil", Earthqu. Engg and Struct. Dyn., Vol. 20 (1991), 961 - 977.

Vrettos, Ch., B. Prange (1990), "Evaluation of In-Situ Effective Shear Modulus from Dispersion Measurements", Proc. ASCE, Vol. 116, No. GT 10, Oct. 1990.

Waas, G. (1972), "Linear two-dimensional analysis of soil dynamics problems in semi-infinite layered media", Ph. D. Dissertation, University of California, Berkely, Calif.

Wolf, J.P., P. Obernhuber (1982a), "Free Field Response from Inclined SH-Waves and Love-Waves", Earthqu. Engg and Struct. Dyn., Vol. 10, 1982, 823 - 845.

Wolf, J.P., P. Obernhuber (1982b), "Free Field Response from Inclined SV and P-Waves and Rayleigh-Waves", Earthqu. Engg and Struct. Dyn., Vol. 10, 1982, 847 - 869.

Wood, A.B. (1930), "A Textbook of Sound", G. Bell and Sons, London

Woods, R.D. (1967), "Screening of Elastic Surface Waves by Trenches", Ph. D. Dissertation, Univ. of Michigan, Ann Arbor, Mich.

Woods, R.D. (1968), "Screening of Surface Waves in Soils", Proc. ASCE, Vol. 94, No. SM4, July.

Woods, R.D., N.E. Barnett, R. Sagesser (1974), "Holography - A New Tool for Soil Dynamics", Proc. ASCE, No. GT11, Nov.

Woods, R.D., K.H. Stokoe II (1985), "Shallow Seismic Exploration in Soil Dynamics", Richart Commemorative Lectures, Publication of the ASCE (Woods), Oct. 1985.

Wu, S., L. Chen (1991), "Dispersion Characteristics of Elastic Waves in Saturated Soils", Proc. 2nd Int. Conf. on Rec. Adv. in Geotech. Earthqu. Engg and Soil Dyn., St. Louis, Missouri.

Xian-Jian, Y. (1991), "Ground Vibration Isolation by Silo and Pile Barriers", Proc. 2nd Int. Conf. on Rec. Adv. in Geotech. Earthquake Engg and Soil Dyn., St. Louis, Missouri.

Yu, P., F.E. Richart (1984), "Stress Ratio Effects on Shear Modulus of Dry Sands", Proc. ASECE, Vol. 110, No. GT3, March.

Zhao, C., S. Valliappan (1993), "Seismic wave scattering effects under different canyon topographic and geological conditions", Soil Dyn. and Earthqu. Engg, Vol. 12, 129 - 143.

Zoeppritz, K. (1919), "Nachrichten der königlichen Gesellschaft des Wissens zu Göttingen", Math.-Physik, Berlin 1919.