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Vibra

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Summary: Construction-induced vibrations may be detrimental to adjacent structures and sensitive electronics operating nearby. Construction vibration sources have a wide range of energy and velocit function of time, transmitted on the ground. Analysis of existing methods for predicting ground and s vibrations shows that empirical equations provide calculations only of amplitudes of vertical soil vibı with insufficient accuracy. This paper presents the application of the impulse response function conc solve the geotechnical problem of predicting ground and structure vibrations before installation of vil sources. Impulse response functions reflect real behavior of soil and structures without the investigat soil and structure properties. A procedure is presented to compute predicted ground and structure vib Good agreement is found between predicted and measured records.

1. Introduction

Sources of construction vibration, such as pile driving, dynamic compaction, blasting and operation c construction equipment, may harmfully affect surrounding buildings and its effect ranges from seriou disturbances of working conditions for sensitive devices and people to visible structural damage.

Considerable data have been collected, analyzed and published with respect to vibrations from constr and industrial sources, *e.g.* Barkan (1962), Richart et al. (1970), Wiss (1981), Mayne (1985), Massar (1992), Svinkin (1993), Dowding (1996), Woods (1997) and others.

Empirical equations employed for assessment of expected soil vibrations from construction and indu sources usually only allow calculation of a vertical peak amplitude of vibrations and not always with sufficient accuracy. These equations cannot incorporate specific differences of soil conditions at each because heterogeneity and spatial variation of soil properties strongly affect characteristics of propag waves in soil from construction and industrial vibration sources.

Svinkin (1996a; 1997) has originated the Impulse Response Function Prediction method (IRFP) for determining complete time domain records on existing soils, structures and equipment prior to install construction and industrial vibration sources. The IRFP method has significant advantages in compar with empirical equations and analytical procedures.

The purpose of this paper is to discuss various approaches and their accuracy to predict and calculate structure vibrations before the beginning of construction activities.

2. Construction Vibrations

2.1. Sources of Vibrations

Impact hammers are common sources of construction vibrations. Maximum rated energy of the most commonly employed piling hammers varies from 5 to 200 kJ per blow. Two kinds of frequencies are observed on the pile acceleration and velocity records. Vibrations with high frequencies of about 300 are generated by the hammer-cushion system. Soil vibrations with such frequency content should be into account when pile driving occurs in close proximity to a building. Frequencies of natural longitu pile oscillations are in the range of 7-50 Hz, with predominance at the lower values. Measured maxin velocity and displacement values range from 0.9 to 4.6 m/s and 12 to 35 mm, respectively. Both para depend on pile type and hammer transferred energy. Displacement might be affected by soil conditio well (Svinkin, 1992; 1996b).

Vibratory hammers for driving non-displacement piles usually have low to moderate force amplitude operating frequencies above 20 Hz. Displacement piles are driven by vibratory hammers with freque around 10 Hz and commonly along with much higher force amplitudes (Warrington, 1992). The soil resistance to pile penetration and the seismic effect of vibratory driven piles depend substantially on conditions, pile type and vibratory hammer model. A coincidence of the operating frequency with the layer frequency may generate large ground vibrations of the soil surrounding the pile. The use of vib hammers with variable frequency and force amplitude may minimize damage due to accidental grou vibration amplification.

For dynamic compaction of loose sands and granular fills, a large steel or concrete weight of 49.1 to is usually dropped from a height of 15 to 30 m. Such dynamic impacts generate surface waves with a dominant frequency of 3 to 12 Hz. Thus, dynamic deep compaction is also a source of intensive low frequency ground vibrations (Mayne, 1985).

The dominant frequency of propagating waves from quarry and construction blasting ranges mostly b 10 and 60 Hz. Blasting energy is much larger than energy of other sources of construction vibrations. example, the energy released by 0.5 kg of TNT is 5400 kJ (Dowding, 1996). Such energy is 50 to 10 the energy transferred to piles during driving and 15 to 80 times the energy transferred to the ground dynamic compaction of soils.

2.2. Ground Vibrations

Sources of construction vibrations generate compression, shear and Rayleigh waves (Barkan, 1992, F al., 1970). Rayleigh waves have the largest practical interest for design engineers because building foundations are placed near the ground surface. In addition, Rayleigh waves contain roughly 70 % of vibration energy and become predominant over other wave types at comparatively small distances fr vibration source. For example, pile driving from depths between 4 and 10 m will generate Rayleigh v within 0.4 to 3 m of the pile, depending on the propagation velocities of Rayleigh and compression w

Soil vibrations are mostly vertical near the source of vertical impact loads, but as distance increases, and horizontal soil vibrations become similar in magnitudes, and, for some locations at the ground su the amplitude of horizontal vibrations might be up to three times greater than that of vertical vibratio Waves travel in all directions from the source of vibrations forming a series of fairly harmonic waves dominant frequency equal or close to the frequency of the source. Spectra of the radial and transvers components of horizontal soil vibrations may have a few maxima and the one corresponding the freq the source is not always the largest. In general, faster attenuation of high frequency components is th primary cause of changes of soil vibrations with distance from the source. However, some records ca explained by this mechanism and the effect of soil strata heterogeneity and uncertainties of the geolo profile should be taken into account (Svinkin, 1996a).

The proximity of the frequency of horizontal soil vibrations to one of the building's natural frequenc generate the conditions of resonance in that building. Moreover, vertical ground vibrations can cause dangerous structural settlements. Considerable data have been collected and published with respect t intolerable vibrations and settlements from construction and industrial sources, *e.g.* Barkan (1962), R

al. (1970), Wiss (1981); Lacy and Gould (1985); Massarsch (1992), Svinkin (1993); Dowding (1996)
others.

To prevent the unacceptable effect of construction vibrations, it is important to accurately predict exp
ground and structure vibrations.

3. Empirical Equations

3.1. Golitsin Equation

As early as 1912, Golitsin (1912) suggested the following equation to calculate the amplitude reducti
Rayleigh waves, generated by an earthquake, between two points at distances r_1 and r_2 from the sour

$$A_2 = A_1 \sqrt{r_1 / r_2}\, e^{-\gamma(r_2 - r_1)} \qquad (1)$$

Where

A_1 = amplitude of vibrations at a distance r_1 from the source
A_2 = amplitude of vibrations at a distance r_2 from the source
γ = attenuation coefficient.

The term $(r_1/r_2)^{0.5}$ indicates the radiation or geometric damping and the term $\exp[-\gamma(r_2-r_1)]$ indicates
material damping of wave attenuation between two points.

Equation (1) has been originally derived to estimate the attenuation of low frequency Rayleigh waves
large wavelength for which the coefficient, γ, depends slightly on the soil upper layers properties. Fo
conditions, the coefficient, γ, changes reasonably in a narrow range in assessment of attenuation prop
soils.

Subsequently, in some studies, by Barkan (1962), Richart et al. (1970), Massarsch (1992), Woods (19
others, this equation was used for preliminary computation of ground vibrations from industrial and
construction sources. Waves generated in the ground by construction sources have higher frequencies
smaller wavelength in comparison with waves from earthquakes and propagate mostly in the upper s
close to the ground surface. It is obvious that the coefficient, γ, is important for the accurate predictio
wave attenuation. Values of γ for various soil types can be found in the referenced publications. An
experimental study to quantify of the coefficient, γ, was performed by Woods and Jedele (1985) who
investigated soil damping for different construction operations at sites with soils of various stratificat
The observed data were approximated by average curves for frequencies of 5 and 50 Hz.

Nevertheless, there are some other factors that affect the coefficient, γ. Collected experimental data in
(Svinkin, 1973; 1992) that the coefficient, γ, depends on physical parameters related to the vibration
(pile impedance, length and transferred energy to the pile, for example), frequency, distance from the
and variation of soil stratification at a site. Test data along the ground surface shows that for various p
widely separated points on the ground surface, values of γ can differ more than an order of magnitude
even change sign. Thus, the coefficient, γ, acceptable for small distances may be inadequate for long
distances. On account of wave refraction and reflection from boundaries of diverse soil layers, an arb
arrangement of geophones at a site can yield incoherent results of ground vibration measurements be
waveforms measured at arbitrary locations at a site might represent different soil layers. Coherent and
consistent results for assessment of surface wave attenuation can be obtained on the basis of measure
ground vibrations reflected from the same soil layer boundaries. Heisey et al. (1982) indicated certain

requirements for a choice of appropriate spacing of the receivers in the application of the spectral ana surface waves (SASW) method in the evaluation of soil properties.

One more important point. If it is possible to measure an amplitude of ground vibrations at the refere distance r_1 during construction activities, a ground vibration amplitude at a distance r_2 can be measu well. Besides, actual structure responses to ground vibrations can be measured without any predictio expected vibrations. In the correct prediction of ground vibration amplitudes before the beginning of construction operations, the referenced amplitude is usually unknown. So, problems with uncertainty assignment or determination of the coefficient, γ, and unknown referenced amplitude show that equa cannot be used for predicting ground vibrations and the application of equation (1) to calculate of gro vibrations from construction and industrial sources may yield inaccurate results.

3.2. Scaled-Distance Approach

The scaled-distance approach, ground velocity-distance-energy relationship, was proposed by Attwel Farmer (1973) to calculate the peak ground velocity at surface distance, D, from a source normalized energy as

$$v = k[D/\sqrt{W_r}\,]^{1} \quad (2)$$

Where

W_r = energy of source or rated energy of impact hammer
k = value of velocity at one unit of distance.

Wiss (1981) reported an identical equation

$$v = k[D/\sqrt{W_r}\,]^{-n} \quad (3)$$

Where the value of 'n' yields a slope in a log-log plot between 1.0 and 2.0 with an average value of 1. an important finding because a slope of amplitude attenuation for all tested soils was in the narrow ra to 2. It turned out that the scaled-distance approach was very useful in the assessment of construction vibrations. Woods and Jedele (1985) and Woods (1997) gathered data from field construction project which the source energy was known or could be estimated and developed a scaled distance chart that correlated with ground types. Most of these data correlated with a slope of n=1.525 for soil class II a of the data presented in that study showed n=1.108 for soil class III.

On the basis of the actual range of energy transferred to piles and the range of the measured peak par velocity at the top of steel, concrete and timber piles, the results of Woods and Jedele (1985) were ad Svinkin (1992; 1996b) to calculate the peak ground velocity prior to the beginning of pile driving. Th vertical ground velocity versus scaled distance from driven piles is depicted in Fig. 1. The reasonable velocity range for steel, concrete, and timber piles is 4.6 to 2.4, 2.4 to 0.9 and 4.6 to 1.5 m/s, respecti latter is actually the same as for steel piles. Values of 4600, 2400 and 900 mm/s have been marked as left values on the slope lines. There are two areas constructed on the diagram: the upper area for steel timber piles and the lower one for concrete piles with a slope, n=1.000, which is the upper limit for t particle velocity with the lower value for the rate attenuation. Data presented in Fig. 1 provide an opp to construct curves of the expected maximum peak ground velocity for various distances from pile dr sources and different magnitudes of transferred energy. The peak particle velocity at the pile head ca calculated in advance as (Svinkin, 1996b)

$$v = \sqrt{2\frac{c}{ZL}W_t} \qquad (4)$$

where

Z = ES/c is pile impedance
E = modulus of elasticity of pile material
S = pile cross-sectional area
c = velocity of wave propagation in pile
W_r = energy transferred to the pile.

This new development of the scaled-distance approach eliminates the need to know in advance the fa
and enhances accuracy of predicted peak ground velocity before pile installation.

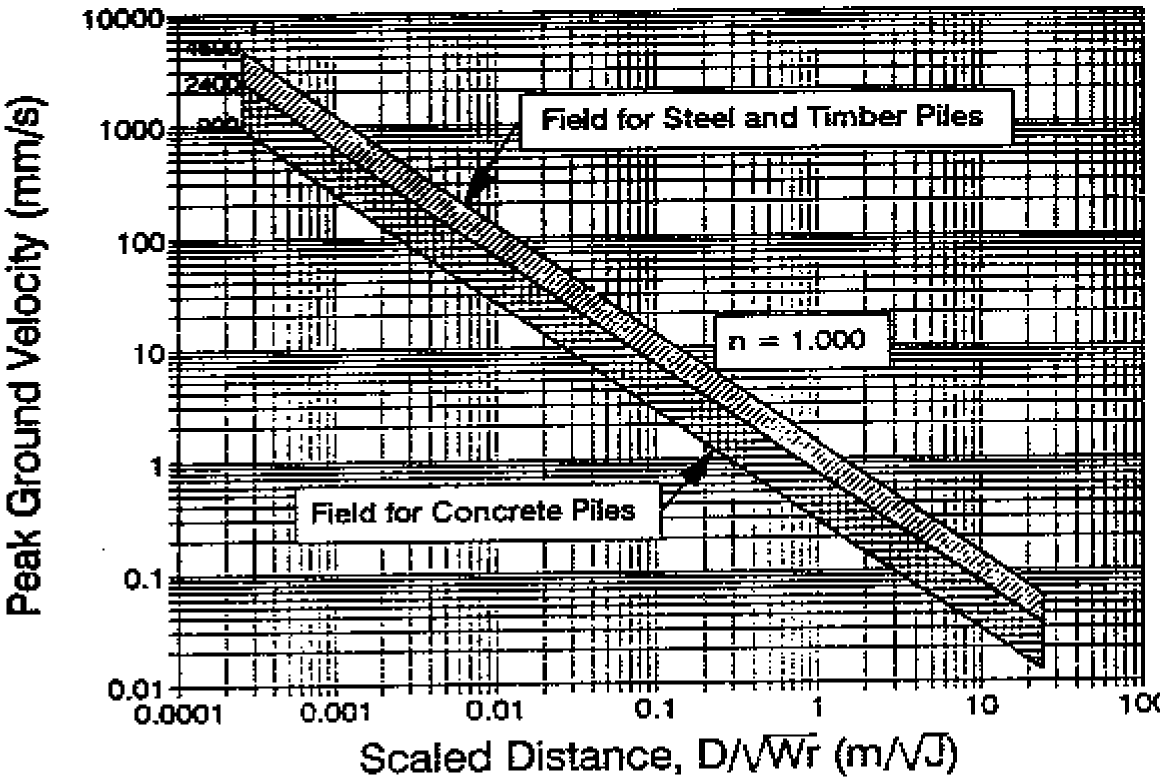
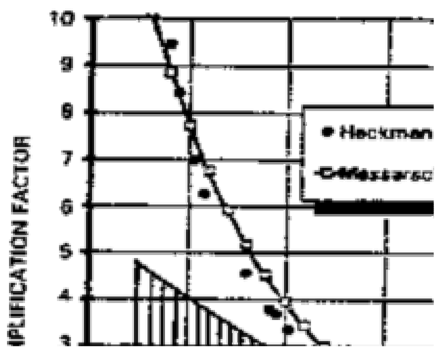


Figure 1. Peak Ground Velocity versus Scaled Distance for Pile Driving

3.3. Pile Impedance

Heckman and Hagerty (1978) and Massarsch (1992) pointed out the important effect of the pile impedance on the peak ground velocity and showed that a reduction of the pile impedance from 2000 to 500 kNs/m could increase the peak ground velocity by a factor of 8 (Fig. 2). According to equation (4), the peak particle velocity of the source is inversely proportional to the square root of the pile impedance and, for the referenced impedance range, the expected amplification of the peak pile velocity and the peak ground velocity can only be 2. Equation (4) shows that pile length, velocity of wave propagation in the pile, and transferred



energy also can affect the peak ground velocity by means of the wave source velocity.

The analysis of soil vibration records, measured at the same distances from a few piles with different impedances and driven by the same hammer to the same pile penetration into the ground, was conduc turned out that a certain range represents the effect of the pile impedance on ground vibration velocit than a single line (Fig. 2). A lower boundary of this range can be calculated using Equation (4).

3.4. Frequency of Vibration

Frequency and peak particle velocity are basic parameters for assessment of ground vibrations. Dow (1996) underlying the importance of frequency because structural responses depend on the frequency ground vibrations. The dominant frequency of expected ground vibrations can be determined prior to beginning of pile driving (Svinkin, 1992).

4. Predicting Structure Vibrations - IRFP Method

4.1. Proposed Approach

The impulse response function prediction method (IRFP) is based on the utilization of the impulse re function technique for predicting complete vibration records on existing soils, buildings and equipme to installation of construction and industrial vibration sources (Svinkin, 1996a; 1997). The impulse re function (IRF) is an output signal of the system based on a single instantaneous impulse input. Impul response functions are applied in the analysis of any complicated linear dynamic system with unknov internal structure for which its mathematical description is very difficult. In the case under considerat dynamic system is the soil medium through which waves propagate outward from sources of constru industrial vibrations. The input of the system is the ground at the place of pile driving, dynamic com of soil, or installation of a machine foundation; the output is a location of interest situated on the surf inside the soil, or any point at a building subjected to vibrations. Outcomes can be obtained, for exan the vibration records of displacements or velocities at locations of interest.

Impulse response functions of the considered dynamic system are determined by setting up an experi (Fig. 3). Such an approach (*a*) does not require routine soil boring, sampling, or testing at the site wh waves propagate from the vibration source, (*b*) eliminates the need to use mathematical models of so profiles, foundations and structures in practical applications, and (*c*) provides the flexibility of consic heterogeneity and variety of soil and structural properties. Unlike analytical methods, experimental I reflect real behavior of soil and structures without investigation of the soil and structure properties. B of that, the proposed method has substantially greater capabilities in comparison with other existing r

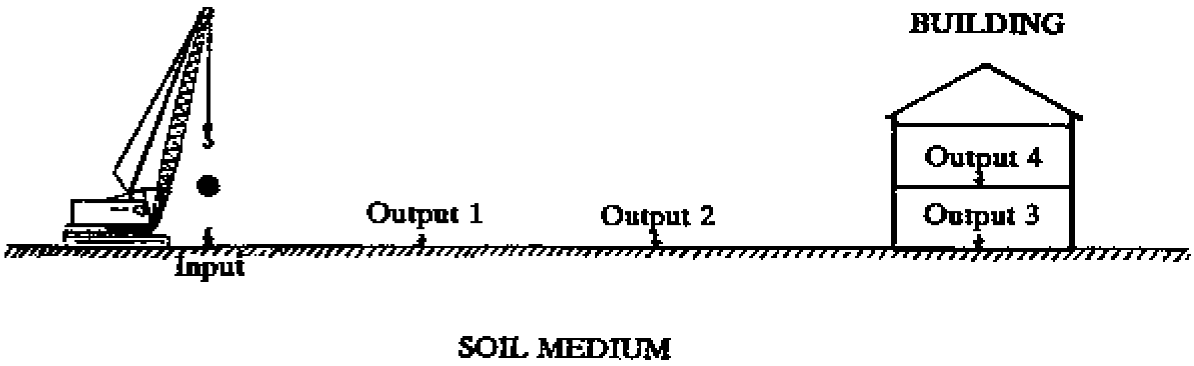


Figure 3. Experimental Determination of Impulse Response Functions

The following is a general outline of the method for predicting vibrations at a distance from an impac

1. At the place in the field for installation of the impact source, impacts of known magnitude are a on the ground (Fig. 3). The impact can be created using a rigid steel sphere or pear-shaped mas from a bridge or mobile crane or a hammer blow on the tested pile. At the moment of impact o ground, oscillations are measured and recorded at the points of interest, for example, at the loca devices sensitive to vibrations. These oscillations are the IRFs of the considered system which automatically take into account complicated soil conditions.
2. Various ways are used to determine the dynamic loads on the ground from different vibration s For pile driving, dynamic loads are computed by the wave equation analysis. In the case of the operation of machines on foundations, these loads can be found using existing foundation dyna theories. For dynamic compaction sites, loads from the source are easily calculated with known weights and heights.
3. Duhamel's integral (Smith and Domney, 1968) is used to compute predicted vibrations which v from operating construction impact source.

4.2. Dynamic Loads Onto Ground

Machine Foundations

Dynamic loads at a machine foundation can be found using existing foundation dynamics theories, fo example Barkan (1962) and Richart et al. (1970). It is known that the equation of vertical damped vib of foundations for machines with dynamic loads can be written as

$$\ddot{z}(t) + 2\alpha\dot{z}(t) + f_{nz}^2 z(t) = p(t) \quad (5)$$

with

$$2\alpha = \frac{b}{M};\ f_{nz}^2 = \frac{k_z}{M};\ p(t) = \frac{P(t)}{M} \quad (6)$$

where

b = viscous damping coefficient, kN/m/s
k_z = spring constant for the vertical mode of foundation vibrations, kN/cm
P(t) = exciting force, kN
M = mass of foundation and machine, t
f_{nz} = circular natural frequency of vertical vibrations of foundation, rad/s
= effective damping constant, rad/s.

An expression derived from equation (5) for a dynamic load applied to the soil is

$$F(t) = M[2\alpha\dot{z}(t) + f_{nz}^2 z(t)] \quad (7)$$

The dynamic force transmitted from the machine foundation to the soil depends on foundation and m masses, the damping constant, the natural frequency of vertical foundation vibrations and vertical fou displacements as a function of time.

Vibration displacements of the machine foundation can be assigned by sampling an arbitrary functio analytically as a damped sinusoid

$$z(t) = \frac{I_F}{M f_{nd}} e^{-\alpha t} \sin(f_{nd} t) \quad (8)$$

with

$$\alpha = \frac{\Phi f_{nz}^2}{2};\ f_{nd}^2 = f_{nz}^2 - \alpha^2;\ f_{nz}^2 = \frac{k_z S_1}{M} \quad (9)$$

where

- I_F = impulse force transmitted from machine to foundation, kNs
- = modulus of damping, s/rad
- $k_{z¢}$ = coefficient of vertical subgrade reaction, kN/m^3
- f_{nd} = circular natural frequency of vertical damped vibrations of foundation, rad/s
- S_1 = contact area between foundation and soil, m^2.

The modulus of damping, Φ, ranges in a relatively narrow range and is slightly dependent on soil cor (Savinov, 1979). For instance, values of Φ range from 0.004 to 0.008 s/rad for the foundation contact less than 10.0 m2. Coefficient, $k_{z¢}$ is determined according to Barkan (1962). Also, it is possible to u approaches for determining values of α and $k_{z¢}$.

Pile driving

Equation (7) can be used to determine the dynamic loads transferred from the pile to the surrounding this case, M is the pile mass. The effective damping constant, α, is chosen from the range of damping constants for foundations with the smallest contact areas. A frequency of the hammer-pile-soil systen calculated by an equation (Svinkin, 1992) which takes into account pile material, the ratio of wave ve the pile to pile length and the pile weight to ram weight ratio.

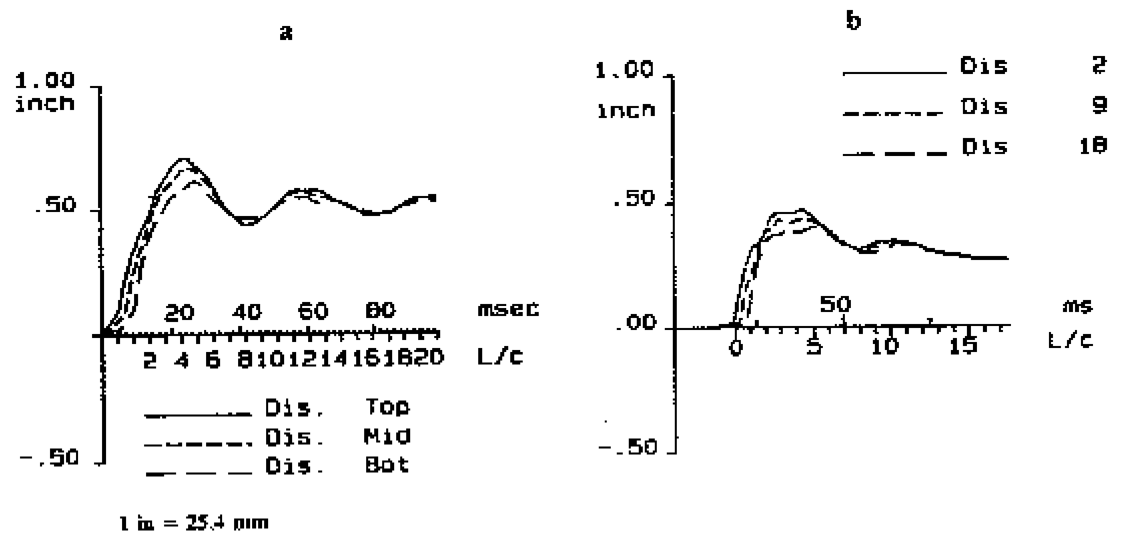


Figure 4. Displacements at Pile Top, Middle and Bottom during Driving: a - Compute GRLWEAP, b - Measured at Pile Top and Obtained by CAPWAP Analysis at Pile Mi and Bottom

Pile displacements as a function of time are computed by the wave equation analysis, using for exam
GRLWEAP Program (GRL and Associates, 1995). Computed displacement records at the pile top, m
and bottom are presented in Fig. 4 for a 457 mm x 457 mm prestressed concrete pile with a length of
These three records are very similar and displacements at the pile top can be taken as a function z(t).
comparison, pile displacements at the same points, obtained by dynamic measurement at the pile top
computed with the CAPWAP program (GRL and Associates, 1993) for the pile middle and bottom a
in Fig. 4. Both sets of curves were derived for the same pile capacity. It can be seen that measured re
confirm the reasonableness of the use of the wave equation analysis to compute pile displacements fo
vibration predicting.

Dynamic Compaction

For dynamic compaction of granular soil, loads from the source are calculated with known falling we
and heights.

4.3. Computation of Predicted Vibrations

For each single output point, the considered input - soil medium - output system is a one degree of fr
system and predicted displacements can be written as follows

$$Z(x, y, t) = \int_0^t F(\tau)\, h_z(x, y, t-\tau)\, d\tau \qquad (10)$$

where F(τ) = the resultant dynamic force transmitted to the ground; x,y = coordinates of the output po
under consideration at the ground or the structure; $h_z(x,y,t-\tau)$ = impulse response function at the outp
under consideration; = variable of integration.

With substitution of expressions (8) and (9) equation (10) becomes

$$Z(t) = \frac{I_F}{f_{nd}} \int_0^t [(f_{nz}^2 - 2\alpha^2)\, e^{-\alpha\tau} \sin(f_{nd}\tau) + 2\alpha f_{nd}\, e^{-\alpha\tau} \cos(f_{nd}\tau)]\, h_z(t-\tau)\, d\tau \qquad (11)$$

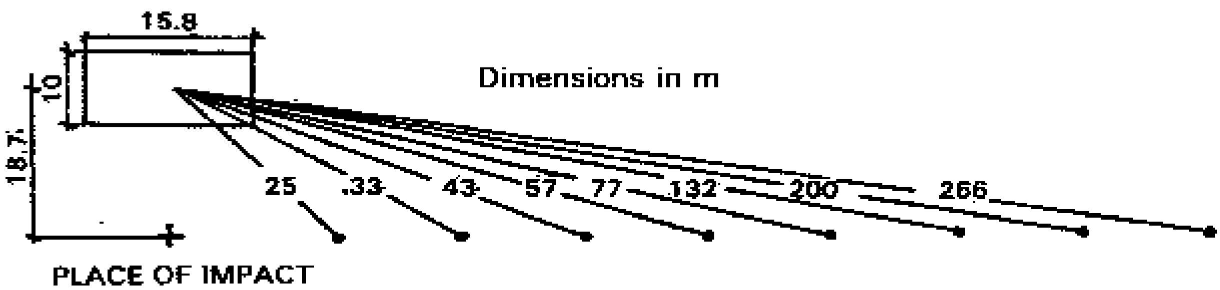


Figure 5. Layout of Machine Foundation, Place of Impact on Ground and Geoph

Examples of predicted results are shown in Fig. 5, 6 and 7. Measurements and prediction of vertical a
horizontal ground surface displacements were made at diverse distances from the foundation under a
drop hammer with a falling weight of 147.2 kN and a maximum drop height of 30.0 m. The soil at th

consisted of about 1.6 m of loose sand followed by about 6.8 m of medium density sand and 1 m of s clay underlain by about 10 m of slightly moist sand. The water table was about 6 m below the grounc The Rayleigh wave velocity was 270 m/sec. A layout of the machine foundation, the place of impact ground and geophones is displayed in Fig. 5.

Predicted and measured vertical and horizontal components of ground surface vibrations at eight loca shown in Fig. 6. It can be seen that good agreement is matched in time domain vibration records, exc horizontal vibrations at two locations close to the foundation. This can be explained by the different paths from the foundation under the operating machine and the place for impact on the ground. The c between these two sources was 18.7 m. Lack of coincidence of the two dynamic sources slightly affe predicted ground vibrations at a distance from the machine foundation. Agreement of predicted and r vibration displacements is quite satisfactory. The differences between the peak predicted and measur vibration amplitudes are less than 30 % at distances larger than 43.0 m from the foundation (Table 1) some individual points amplitudes actually coincide.

Table 1. Peak Measured and Predicted Vibration Amplitudes

Distance from source (m)	Vertical			Horizontal		
	Measured (µm)	Predicted (µm)	Error (%)	Measured (µm)	Predicted (µm)	Error (%)
25	450	330	-27	180	510	+183
33	351	216	-38	227	396	+74
43	270	232	-14	238	252	+6
57	-	-	-	162	144	-11
132	55	60	+9	-	-	-
200	30	30	0	65	59	-9
266	28	36	+30	25	29	+16

Spectrum analysis of predicted and measured time histories revealed that both records have similar fr domain curves with the same dominant frequency. Moreover, predicted records are slightly dependen parameters in equation (7) for determination of the dynamic force transmitted from the source to the (Svinkin, 1999).

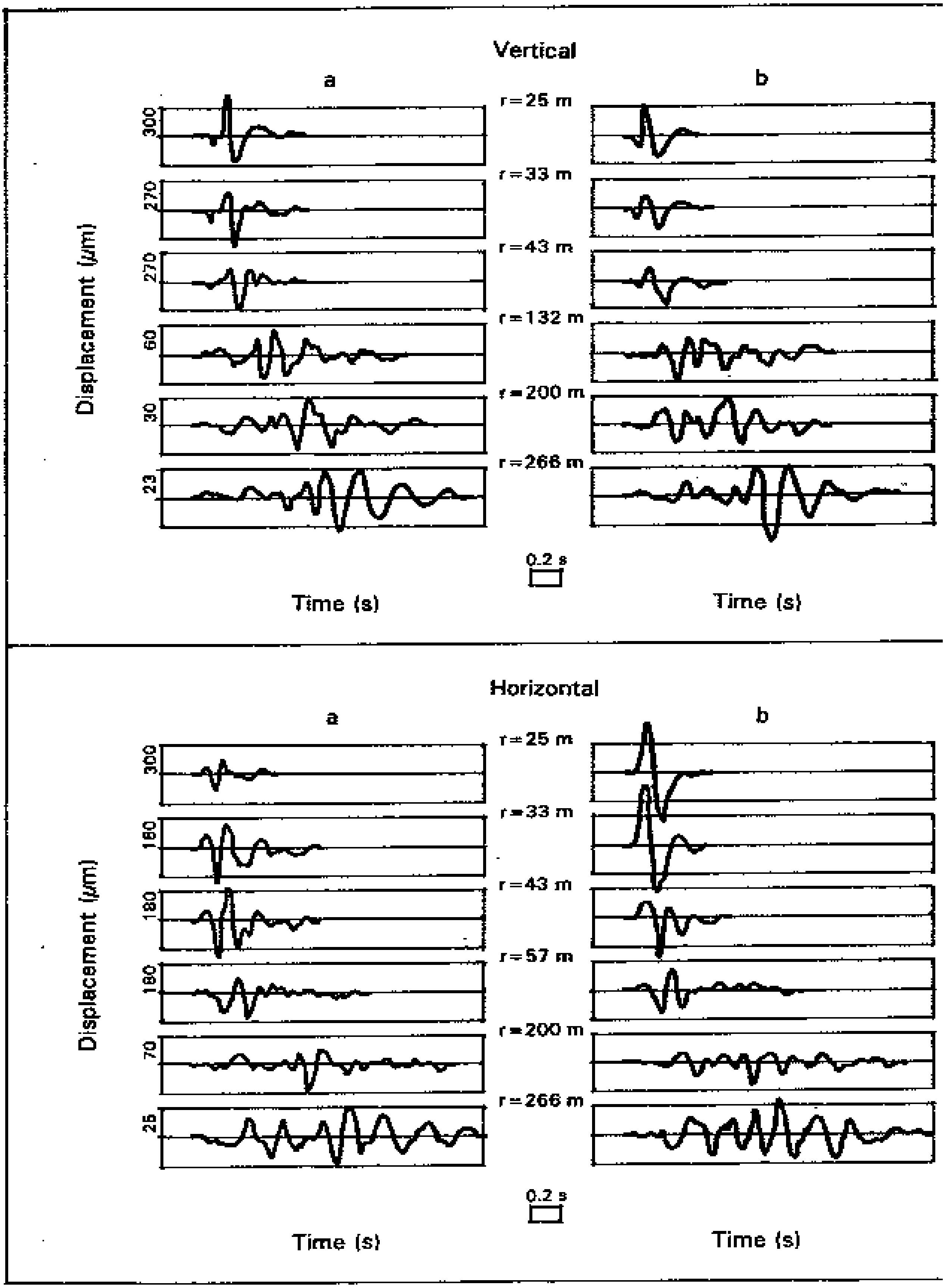


Figure 6. Vertical and Horizontal Soil Vibrations from Operating Drop Hammer, Measur and Predicted (b) Vibrations

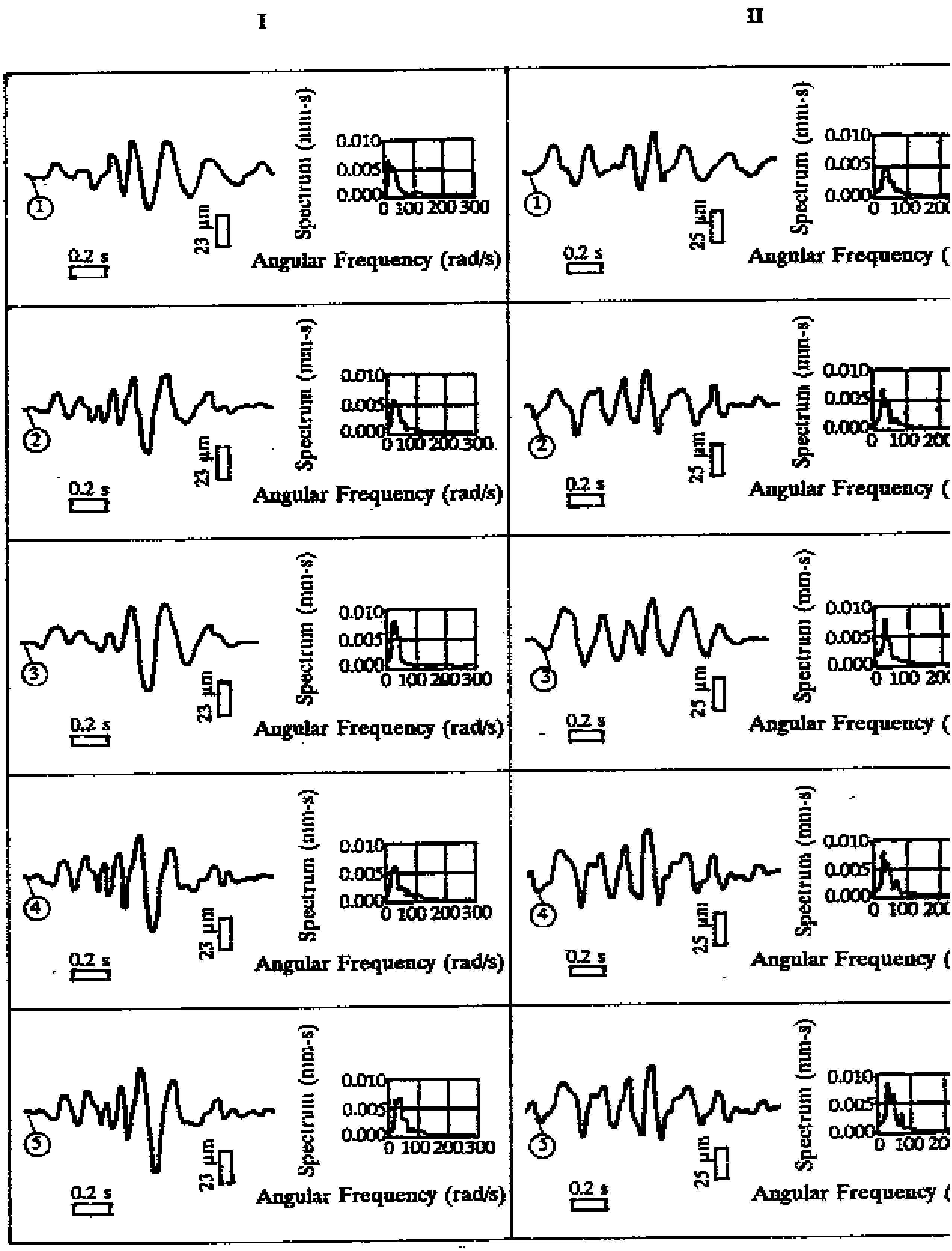


Figure 7. Records and Spectra of Vertical and Horizontal Soil Vibrations at 266.0 m Foundation of Drop Hammer; ① - Measured Vibrations; ② - ⑤ Predicted Vibratio Various Initial Parameters defined in Table 2

Predicted vibration curves in Fig. 7 at a distance of 266.0 m from the machine foundation were comp with various values of initial parameters in Eq. (7), Table 2. In spite of the change of the computed n foundation frequency in the range of 23.8-63.5 rad/s and the damping constant from 8.5 to 60.5 rad/s shapes of measured and predicted records are almost the same and their spectra show the same domi vibration frequency. An increase of the computed natural frequency of foundation vibrations with res the measured frequency leads to an increase of the largest amplitude by 10-30 % for both vertical and horizontal predicted soil oscillations. Spectra of these oscillations show a stability of frequency comp for even very long duration soil oscillations. Thus, variations of predicted soil oscillations do not exc measurement errors even with a 2.7 times increase in the computed natural frequency of the foundati

TABLE 2. Parameters of Foundation-Soil System

Record No.	k_z (kN/m^3)	α (rad/s)	Φ (s/rad)	f_{nz} (rad/s)	M t
2	Experimental time domain foundation displacement				
3	34433	8.5	0.03	23.8	9614
4	67885	60.5	0.03	63.5	2650
5	39240	35.0	0.03	48.3	2650

5. Conclusions

- Construction operations such as pile driving, dynamic compaction and blasting are wide-spread of ground and structure vibrations. These vibration sources have a wide range of energy and ve as a function of time, transmitted on the ground. Construction-induced vibrations may harmful surrounding buildings. It is important to accurately predict vibrations of ground, structures, and sensitive devices prior to the beginning of construction activities to avoid the undesirable effec generated vibrations.
- Empirical equations provide only calculation of a vertical amplitude of ground vibrations and r always with sufficient accuracy. For pile driving, the scaled distance approach with calculated particle velocity of the source is probably the most appropriate method for predicting upper lin the peak particle velocity of ground vibrations. The effect of pile impedance on ground vibrati exaggerated in some publications. Other parameters of the hammer-pile-soil system like pile le velocity of wave propagation in the pile, and transferred energy to the pile can affect the peak g velocity as well.
- The impulse response function prediction method (IRFP) is used to solve a geotechnical proble predicting time domain ground and structure vibrations prior to the beginning of construction a or installation of machine foundations.
- The proposed approach uses the impulse response function technique for a considered dynamic ground at the place for the source of vibrations - soil medium - output locations of interest on t ground or in any structure receiving vibrations. Experimental impulse response functions refle soil behavior and take into account uncertainty in the geologic environment. Such an approach require routine soil boring, sampling, and testing at the site where waves propagate from the vi source. Different ways were shown to determine dynamic loads onto the ground from machine foundations, pile driving and dynamic compaction of granular soil. An algorithm is presented t compute predicted vibrations, and examples of predicted results are demonstrated for vertical a horizontal ground displacements. There is quite satisfactory agreement between predicted and measured records.
- The proposed approach provides the method for determining and monitoring of ground, structu sensitive devices vibration levels before the start of construction or industrial vibration activiti

6. Acknowledgement

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