Some Applications of Soil Dynamics

The 2009 Spencer J. Buchanan Lecture

By Professor Jose M. Roesset

Criteria for Geotextile and Granular Filters

The 2008 Karl Terzaghi Lecture

By Jean- Pierre Giroud

Friday, November 13, 2009
College Station Hilton
College Station, Texas, USA
http://ceprofs.tamu.edu/briaud/buchanan.htm

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SPENCER J. BUCHANAN

Spencer J. Buchanan, Sr. was born in 1904 in Yoakum, Texas. He graduated from Texas A&M University with a degree in Civil Engineering in 1926, and earned graduate and professional degrees from the Massachusetts Institute of Technology and Texas A&M University.

He held the rank of Brigadier General in the U.S. Army Reserve, (Ret.), and organized the 420th Engineer Brigade in Bryan-College Station, which was the only such unit in the Southwest when it was created. During World War II, he served the U.S. Army Corps of Engineers as an airfield engineer in both the U.S. and throughout the islands of the Pacific Combat Theater. Later, he served as a pavement consultant to the U.S. Air Force and during the Korean War he served in this capacity at numerous forward airfields in the combat zone. He held numerous military decorations including the Silver Star.

He was founder and Chief of the Soil Mechanics Division of the U.S. Army Waterways Experiment Station in 1932, and also served as Chief of the Soil Mechanics Branch of the Mississippi River Commission, both being Vicksburg, Mississippi.

Professor Buchanan also founded the Soil Mechanics Division of the Department of Civil Engineering at Texas A&M University in 1946. He held the title of Distinguished Professor of Soil Mechanics and Foundation Engineering in that department. He retired from that position in 1969 and was named professor Emeritus. In 1982, he received the College of Engineering Alumni Honor Award from Texas A&M University.

He was the founder and president of Spencer J. Buchanan & Associates, Inc., Consulting Engineers, and Soil Mechanics Incorporated in Bryan, Texas. These firms were involved in numerous major international projects, including twenty-five RAF-USAF airfields in England. They also conducted Air Force funded evaluation of all U.S. Air Training Command airfields in this country. His firm also did foundation investigations for downtown expressway systems in

Milwaukee, Wisconsin, St. Paul, Minnesota; Lake Charles, Louisiana; Dayton, Ohio, and on Interstate Highways across Louisiana. Mr. Buchanan did consulting work for the Exxon Corporation, Dow Chemical Company, Conoco, Monsanto, and others.

Professor Buchanan was active in the Bryan Rotary Club, Sigma Alpha Epsilon Fraternity, Tau Beta Pi, Phi Kappa Phi, Chi Epsilon, served as faculty advisor to the Student Chapter of the American Society of Civil Engineers, and was a Fellow of the Society of American Military Engineers. In 1979 he received the award for Outstanding Service from the American Society of Civil Engineers.

Professor Buchanan was a participant in every International Conference on Soil Mechanics and Foundation Engineering since 1936. He served as a general chairman of the International Research and Engineering Conferences on Expansive Clay Soils at Texas A&M University, which were held in 1965 and 1969.

Spencer J. Buchanan, Sr., was considered a world leader in geotechnical engineering, a Distinguished Texas A&M Professor, and one of the founders of the Bryan Boy's Club. He died on February 4, 1982, at the age of 78, in a Houston hospital after an illness, which lasted several months.

The Spencer J. Buchanan '26 Chair in Civil Engineering

The College of Engineering and the Department of Civil Engineering gratefully recognize the generosity of the following individuals, corporations, foundations, and organizations for their part in helping to establish the Spencer J. Buchanan '26 Professorship in Civil Engineering. Created in 1992 to honor a world leader in soil mechanics and foundation engineering, as well as a distinguished Texas A&M University professor, the Buchanan Professorship supports a wide range of enriched educational activities in civil and geotechnical engineering. In 2002, this professorship became the Spencer J. Buchanan '26 Chair in Civil Engineering.

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1993	Ralph B. Peck	"The Coming of Age of Soil Mechanics: 1920 - 1970"
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2001	Robert D. Holtz	"Geosynthetics for Soil Reinforcement"
2002	Arnold Aronowitz	"World Trade Center: Construction, Destruction, and Reconstruction"
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2007	Ricardo Dobry	"Pile response to Liquefaction and Lateral Spreading: Field Observations and Current Research"
2008	Kenneth Stokoe	"The Increasing Role of Seismic Measurements in Geotechnical Engineering"
2009	Jose M. Roesset	"Some Applications of Soil Dynamics"

The text of the lectures and a videotape of the presentations are available by contacting:

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AGENDA

The Seventeenth Spencer J. Buchanan Lecture
Friday November 13, 2009
College Station Hilton

2:00 p.m.   Welcome by Jean-Louis Briaud

2:10 p.m.   Introduction by John Niedzwecki

2:15 p.m.   Introduction of Jean-Pierre Giroud by Chloe Arson

2:20 p.m.   "Criteria for Geotextile and Granular Filters"
                 The 2008 Terzaghi Lecture by Jean-Pierre Giroud

3:30 p.m.   Introduction of Jose Roesset by Marcelo Sanchez

3:35 p.m.   "Some Applications of Soil Dynamics"
                 The 2009 Buchanan Lecture by Jose Roesset

4:25 p.m.   Discussion

4:40 p.m.   Closure with Philip Buchanan

5:00 p.m.   Photos followed by a reception at the home of Jean-Louis and Janet Briaud.

José M. Roesset

As a faculty member in the Civil Engineering Department of MIT (1964-1978) Dr Roesset conducted roughly half of his research on Nonlinear Structural Dynamics, with special emphasis on Earthquake Engineering, and the other half on what is known now as Geotechnical Earthquake Engineering. His structural work involved studies on inelastic response spectra, development of nonlinear structural models such as the fiber model, assessment of the validity of approximate procedures to derive equivalent inelastic single degree of freedom systems from incremental nonlinear static analyses of frames (later called the push-over method), and development of formulations in time and frequency domains. His work on geotechnical engineering involved first studies of the effect of local soil conditions on the characteristics of earthquake motions (soil amplification) for different types of seismic waves, then the determination of the dynamic stiffness of mat foundations and single piles, and finally the study of the effects of the soil/foundation flexibility on the seismic response of structures (soil structure interaction). Much of this work found applications in the seismic analysis and design of Nuclear Power Plants, a hot topic at that particular time, and Dr. Roesset served as a consultant in a number of plants.

At the University of Texas at Austin (1978-1997) Dr. Roesset continued to do some work on nonlinear structural dynamics and on dynamic stiffness of foundations (pile groups in particular) but he devoted most of his research effort to more fundamental wave propagation studies with special application to the nondestructive evaluation of soil deposits and pavement systems. This work was performed in collaboration with Dr. Kenneth H. Stokoe (the sixteenth Buchanan lecturer) and involved on one hand the development of the formulation to interpret the data obtained with the Spectral Analysis of Surface Waves (SASW) method in order to backfigure the variation of soil properties with depth, and on the other the interpretation of the data obtained from Dynaflect and Falling Weight Deflectometer (FWD) tests to determined the elastic properties of pavement layers. The studies in this last case included the evaluation of the effects

of the finite width of the pavement and the relative position of the FWD with respect to the edge, and the assessment of the importance of nonlinear soil behavior under large loads, particularly for flexible pavements.

From 1988, at the University of Texas first and at Texas A&M University since 1997, his research concentrated on the nonlinear dynamic response of deep water offshore platforms and fluid structure interaction effects. This work was conducted for the Offshore Technology Research Center (OTRC), a joint venture between Texas A&M and the University of Texas at Austin with headquarters in College Station. Dr. Roesset was first the research coordinator for the center, then the Associate Director for UT Austin, and finally the Director at Texas A&M.

Over the last five years Dr. Roesset has returned to the areas of Structural and Soil Dynamics with studies on the seismic response of base isolated bridges including soil structure interaction effects, the dynamic response of pile foundations with large numbers of piles, and the in situ determination of nonlinear soil properties (work conducted in collaboration with Dr. Kenneth H. Stokoe in Austin and Dr. Giovanna Biscontin at TAMU, under a NEES grant.

SOME APPLICATIONS OF SOIL DYNAMICS

The Seventeenth Buchanan Lecture

Presented by Professor Jose M. Roesset
Texas A&M University

Background.

Soil Dynamics is the branch of Soil Mechanics (or in more fashionable modern terms Geotechnical Engineering) that studies the behavior of soil deposits and earth structures subjected to dynamic loads. It originated in the first quarter of the 20th century with the need to understand and eliminate the vibrations of foundations caused by heavy rotating machinery. It has become since an essential component of Earthquake Engineering (recognized even by some structural engineers), and it has found a number of other important practical applications.

The solution of dynamic problems requires in principle a solid understanding of the behavior of soils under all types of static loads. Yet the static and dynamic fields evolved initially in parallel and separately. It is interesting that while the traditional static counterpart considered always large deformations and proceeded with the use of somewhat arbitrary parameters to characterize the soil on empirical bases, with the development of simplified mechanistic models for design purposes and the belief that soils are too complicated to treat them as elastic (or even inelastic) materials, Soil Dynamics considered small deformations and linear elastic behavior and started from the very early stages with theoretical solutions based on the Theory of Elasticity (or in more fashionable modern terms Elasto-Dynamics) based on the solution of Boussinesq's dynamic problem obtained by Lamb in 1904. An effort was always made however to complement the rigorous theoretical derivations with simplified models that could explain the behavior of the solutions and serve for design purposes, and with experimental data to validate them.

Analytical formulations, whether in closed form, as a series expansion, or in integral form, provide rigorous solutions that are often of direct practical value and that can always be used as benchmarks to judge the validity of simplified procedures, numerical approaches and computer codes. Properly validated numerical models can then provide accurate solutions to real practical problems. To develop however a good feeling and understanding of the physical phenomena, to be able to decide what are the significant parameters that must be known in order to use these models, and to estimate at least orders of magnitude of the results, in order to assess the validity of computer simulations, it is necessary to develop some reliable but simplified procedures. These procedures would be applicable for preliminary studies or design purposes while the more complicated computational models would be used for final verification of very special structures (particularly when all the required parameters such as soil properties and their variation with level of strain are known). It is not surprising hence that the same researchers who developed the more rigorous continuous formulations based on the Theory of Elasticity tried from the very early stages to explain their results using simple models. Any theoretical formulation needs in addition experimental verification before it will be accepted and used in engineering practice. This is so, in some cases, because of a reluctance to accept any fact that cannot be seen with one's own eyes or because of an innate mistrust and aversion towards mathematical derivations. In more enlightened cases, the reluctance stems from the realization that even if a mathematical formulation is correct for a given model it may not include all the variables that influence the physical process and the values of these variables may not be known with certainty. By the same

token no experimental studies can reproduce potentially important effects that are not explicitly accounted for in the test. There is therefore a need to combine and integrate theoretical and experimental research in Soil Dynamics as in any other discipline.

Soil Dynamics encompasses now much more than the original problem of design of machine foundations and, although many geotechnical engineers may still consider it outside of the main stream of the profession, it involves now a large number of different applications. These range from the determination of the dynamic stiffness of different types of foundations as a first step to the analysis of rigid masses or common structures subjected to dynamic loads (machine loads, wind forces, wave action), to the study of the effect of local soil conditions on the characteristics of earthquake motions (soil amplification problem), and seismic soil structure interaction analyses (inertial and kinematic interactions), the seismic response of earth structures (slopes, embankments, levees, dams), the study of vibrations created by construction equipment, such as pile driving machines, or moving loads (particularly subways and high speed trains), and the determination of soil properties in laboratory tests and in situ (geophysical methods based on wave propagation). In this lecture we will look briefly at some of these applications, their historical background, the present state of the art and basic features of the problem, and some of the research needs.

Machine Vibrations

The design of foundations to support heavy machinery that could induce vibrations was already recognized as an important practical problem in the 1920s giving rise to the field of Soil Dynamics. In the thirties Reissner derived the first analytical solution for the vertical displacements on the surface of a linear elastic, homogeneous, and isotropic half space subjected to a harmonic normal stress uniformly distributed over a circular area. Under this assumption the displacements over the circular area would be variable. Reissner selected the value of the vertical displacement at the center of the loaded area as representative of the motion of a rigid massless foundation. The application of these results to study the vibrations of a rigid body resting on soil represents therefore an approximation since the stress distribution under a rigid footing would not be uniform but unknown. In reality the displacement of the foundation would be specified while the stresses would be zero over the remaining free surface of the half space (mixed boundary value problem). The solution for the vertical case was followed immediately by a solution for torsional vibrations. Work along these lines was continued in the following years by Reissner and Sagoci and by Shekhter, who used the average of the displacements at the center and at the edge of the loaded area to obtain curves of dynamic amplification as a function of a dimensionless frequency and a mass ratio. These curves were widely used.

The fifties saw a significant increase both in the number of researchers engaged in this area and in the number of related publications, with important contributions by Arnold, Bycroft, Quinlan, Sung and Warburton among others. Quinlan and Sung considered other stress distributions under the circular footing to assess the effect of this simplifying assumption on the results. Bycroft

accounted for internal soil damping and studied other types of motions. Warburton studied the dynamic response of a rigid foundation on a linear elastic, isotropic and homogeneous soil layer of finite thickness resting on much stiffer rock by opposition to a half space, a case of practical importance that exhibits some marked differences in the solution. Not only will the static stiffness of the foundation be larger (depending on the ratio of its radius to the layer thickness) but a soil layer will have its own natural frequencies leading to larger fluctuations of the stiffness with frequency (the stiffness would become zero at the resonant frequency without internal damping) and a lack of radiation below a threshold frequency.

Additional studies were conducted in the sixties by Borodatchev, Collins, Elorduy, Gladwell, Kobori, Lysmer, Minai, Novak, Richart, Robertson, Sigalov, Stallybrass, and Whitman among others. Of particular practical importance was the publication in 1962 of the English edition of Barkan's book, *Dynamics of Bases and Foundations*, as until then the main books on machine foundations had been either in German (Rausch's various editions of *Machinen Fundamente* and Lorenz's *Grunbau Dynamik*) or in Russian (Barkan's own book first published in 1948).

The solution of the true mixed boundary value problem, where stresses are specified along the surface of the soil, outside the area of the foundation (stress free surface) while displacements are imposed at the base of a rigid and massless body, was addressed by Borodatchev in 1964 for the vertical case. A comprehensive treatment of the problem and an alternative graphical solution for this case were presented by Awojobi and Grootenhuis in 1965, and Lysmer provided a numerical solution the same year. Sigalov extended Boradatchev's work to rocking vibrations; Robertson used a formulation based on a series expansion and Gladwell extended it in 1968. A rigorous solution for the dynamic stiffness of a rigid and massless circular foundation on the surface of a linear elastic, homogeneous and isotropic half space was presented in graphical and tabular form over an extended range of frequencies by Veletsos and Wei for the coupled horizontal and rocking vibrations in 1971 and an independent solution was published by Luco and Westmann almost at the same time. Additional results for vertical and torsional excitations and for viscoelastic or hysteretic media were obtained by Veletsos and Verbic. All these solutions represent important benchmarks and have greatly contributed to our understanding of the behavior of mat foundations under dynamic loads for small amplitude vibrations. Yet there are few soil deposits that can be considered as homogeneous and isotropic half spaces. Elastic moduli of soils will generally vary with depth and there will be some stiffer rock at some depth. With the availability of digital computers and the development of new discrete formulations (finite differences, finite elements, boundary elements) solutions for foundations on the surface or embedded in a horizontally stratified layered soil followed immediately through the work of Waas, Chang-Liang, Kausel, Luco and Dominguez among others. Novak and Beredugo, Kausel, and Elsabee studied the case of circular foundations partially embedded in a soil layer assuming perfect bonding between the lateral walls and the surrounding soil, and suggested approximate formulas for this case. The dynamic stiffness of single piles and pile groups (assuming again linear elastic soil behavior and perfect bonding between the pile and the soil) were investigated

next by Novak, Nogami, Blaney, Kausel, Kaynia, and Gomez. By the late seventies the capability existed to compute the dynamic stiffness of foundations of arbitrary shape in horizontally stratified soil deposits with any desired degree of accuracy as long as linear elastic soil behavior and perfect contact between the foundation and the surrounding soil could be assumed (low amplitude vibrations as might be expected for well designed machine foundations). Spread footings have received however much less attention and a few studies conducted have neglected the interaction between them through the soil making the results questionable, in spite of the fact that the interaction between two neighboring foundations had been studied by Warburton, Richardson, and Gonzalez in the late sixties and seventies.

`At the same time that the analytical formulations were developed Reissner explored the possibility of reproducing his results with a lumped parameter model consisting of a mass, a spring and a dashpot, but he concluded that their values would have to be functions of frequency, and he could not find simple expressions for them. Similar conclusions were reached by others trying to match experimental data with simple models. Shekhter on the other hand, found that the results for the amplification functions in terms of a mass ratio and a dimensionless frequency could be reasonably approximated by a mass-spring-dashpot system. Merritt and Housner substituted the foundation by a rotational spring in their 1954 study of seismic soil-structure interaction: Lycan and Newmark in 1961 replaced the foundation by a free mass. In 1965 Fleming, Screwvala and Kodner used horizontal and rocking springs to simulate interaction effects in swaying and rocking. Lysmer and Richart in 1966, Whitman and Richart in 1967, Hall in 1968, and Whitman in 1969, used again lumped parameter models with springs, masses and dashpots. In the early seventies Meek and Veletsos used a truncated cone to explain successfully some of the basic features of the dynamic stiffness of a foundation and showed that a simple mass-spring-dashpot system was not sufficient to reproduce the exact solution over an extended range of frequencies. For small values of the dimensionless frequency a constant mass, spring and dashpot seem to reasonably reproduce the frequency variation of the stiffness terms and this had given rise to the concept of an added mass of soil vibrating in phase with the foundation. In fact such a model was proposed in a number of technical reports and even widely used books, but Veletsos' studies showed that it is not correct and that lumped parameter models must have frequency dependent terms or be slightly more complicated. Veletsos and Wei and Veletsos and Verbic found approximate but accurate expressions for the foundation stiffness terms of a rigid circular mat on the surface of an elastic, homogeneous and isotropic half space as function of the dimensionless frequency and these are still the best simplified formulas available to date.

The dynamic stiffness of a rigid, or very stiff, mat foundation or of a pile foundation with a very stiff cap can be represented by a 6 by 6 matrix whose terms are complex functions of frequency. The real part of these terms represents the static stiffness and inertial effects in the soil (thus the frequency dependence). The imaginary terms represent the loss of energy (damping) due on one hand to radiation of waves away from the foundation (for all frequencies in the case of a half space or for frequencies higher than a threshold frequency when dealing with a soil layer of finite

thickness), and on the other to internal material damping in the soil (associated with nonlinear soil behavior and function therefore of the level of vibrations). The radiation or geometric damping increases in general with frequency being thus approximately of the viscous type. The internal soil damping would be independent of frequency, of the hysteretic type. For a surface foundation with two planes of symmetry the dynamic stiffness matrix can be considered to be approximately diagonal and the foundation can then be represented by three independent sets of springs and dashpots, still frequency dependent. For an embedded foundation the coupling between horizontal translations and rocking cannot be neglected but one could again in many cases consider the independent sets of springs and dashpots placed at some depth and not at the base. For flexible mat foundations or pile caps this simplified model is no longer applicable and one would have to select a number of points along the contact area between the foundation and the soil and derive a dynamic stiffness matrix with 3 degrees of freedom at each point.

It is common to express the terms of the dynamic stiffness matrix in the form

$$K_{dynamic} = K_{real} + iK_{imaginary} = K_{real} + i\Omega C_{eq} = K_{static}(k_1 + i\frac{\Omega R_{eq}}{c_S}c_1)$$

All the variables are function of the circular frequency in radians/sec Ω, K_{static} is the corresponding static stiffness, C_{eq} is the constant of an equivalent viscous dashpot, R_{eq} is the equivalent radius of the foundation if not circular, c_s is the shear wave velocity of the soil and c_1 and k_1 are the dynamic stiffness coefficients. The presence of an imaginary term implies that the applied dynamic force on the foundation and the resulting displacement are not in phase. Calling φ the phase angle between them $\tan \varphi = K_{imag}/K_{real}$
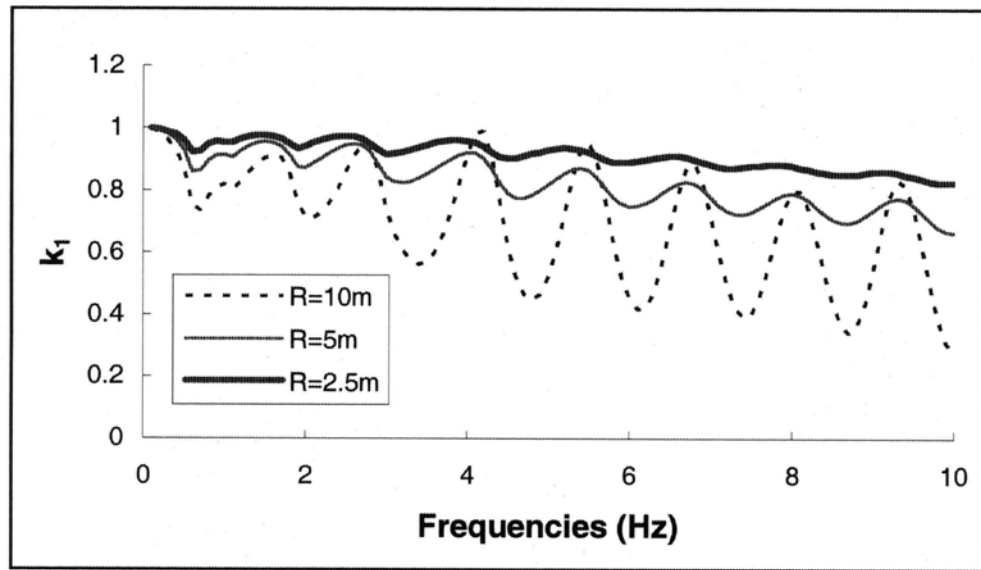

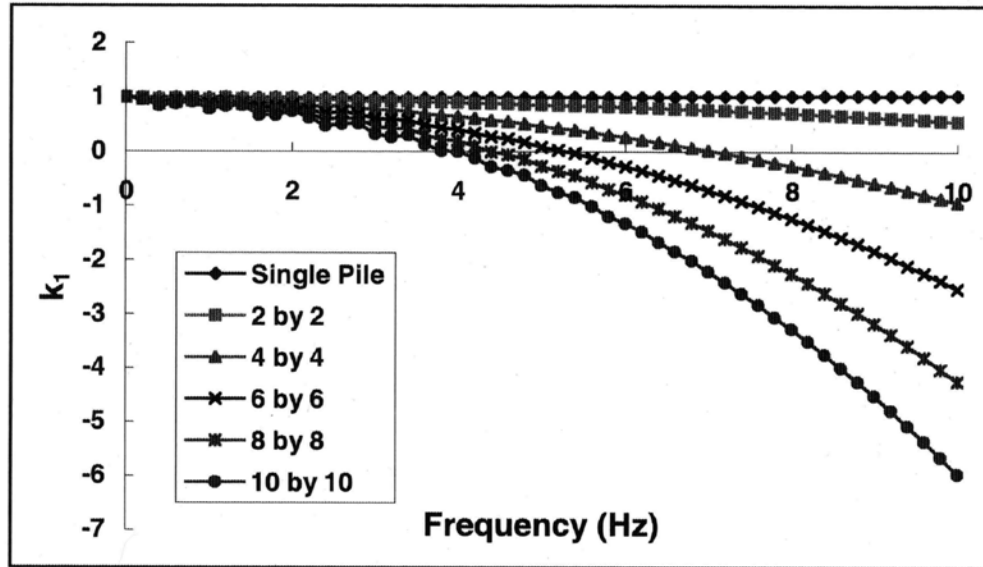Figure 1 k_1 coefficient for horizontal stiffness of mat foundations

Figure 2 k_1 coefficient for horizontal stiffness of pile foundations
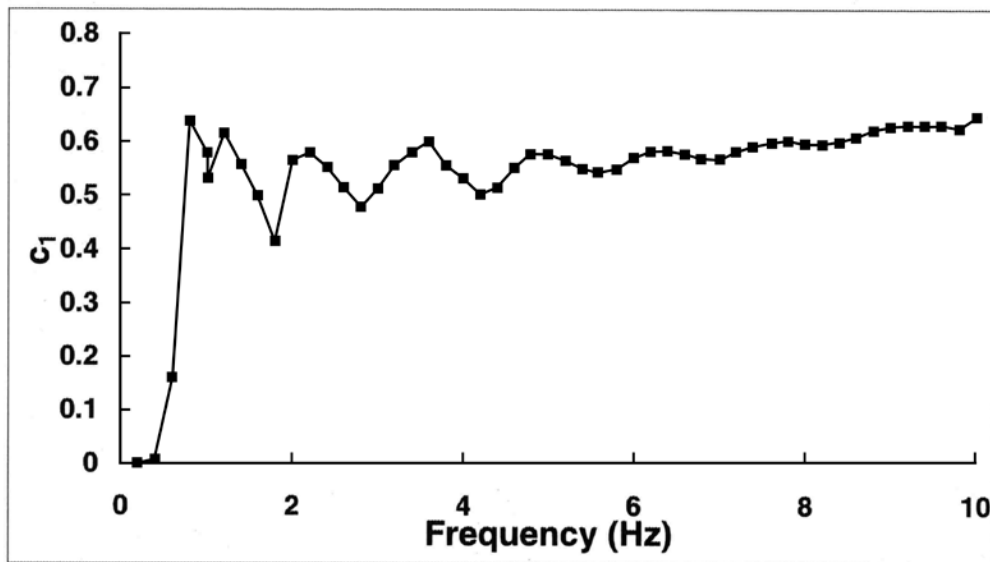

Figure 3 c_1 coefficient for horizontal stiffness of mat or pile foundation

Constant (frequency independent) values of the two dynamic coefficients would imply that the term can be reproduced by a traditional spring and a dashpot. A parabolic variation of k_1 would imply a spring and a mass vibrating in phase with the foundation. Figure 1 shows the real coefficient k_1 and its variation with frequency for the horizontal stiffness of a circular mat foundation resting on the surface of a soil layer of finite depth with 5% internal damping. The results presented are for a stratum of fixed thickness and foundations with different radii. The

oscillations are associated with the natural frequencies of the soil layer. As the radius of the foundation decreases and the ratio of the layer thickness to the radius increases the amplitude of the oscillations decreases and eventually the results approach the solution for a half space. The results for a half space would still show a small variation with frequency but would be much smoother. The coefficient for vertical vibration and rotations (torsion or rocking) would have on the other hand a stronger frequency dependence even for a half space, particularly for values of Poisson's ratio larger than 0.4. Figure 2 shows the corresponding k_1 coefficient for a pile foundation with different number of piles (same pile spacing in all cases). As the number of piles increases the variation of the coefficient tends to approach a second degree parabola suggesting the existence of a soil mass trapped between the piles and vibrating in phase with the foundation. Figure 3 shows the variation of the c_1 coefficient also for the horizontal case. Below the fundamental shear frequency of the layer the coefficient is zero because there can be no radiation. Above that frequency the coefficient shows oscillations around what could be considered a constant value. The threshold frequency under vertical and rocking vibrations is associated with a vertical frequency of the stratum corresponding to the P wave velocity for values of Poisson's ratio equal or lower than 0.3 and with an intermediate wave velocity for larger values of Poisson's ratio. The variation of this coefficient with frequency is larger for rotations (rocking or torsion) than for translations (horizontal or vertical)

When comparing experimental data to theoretical predictions based on published formulas one must take into account that the latter are intended for rigid, massless foundations and for linear elastic homogeneous soils. Including the mass of the test foundation does not represent any problem but it requires the consideration of the response of a single or a two degree of freedom system (depending on the type of excitation) and therefore the comparison requires some additional computations. Through the years many of the researchers above mentioned tried to correlate available experimental data with predictions from the existing theoretical formulations at the time. A number of legitimate reasons were offered for encountered discrepancies, one of the most common being the existence of nonlinear effects. One would expect that for a properly designed machine foundation the dynamic strains induced in the soil by the machine vibrations would be very small, and that therefore linear elasticity would generally apply except at points where there are concentrations of stresses (edges of a mat foundation, near the head of a pile). This will not be always the case for field tests in which different frequencies and amplitudes of excitation are used, or for unbalanced foundations that have been improperly designed. Nonlinear effects are still the main area in need of further research in relation to the dynamic stiffness of machine foundations. It should be noticed however that with important nonlinear effects the response of a foundation to a single frequency harmonic load will no longer be in that frequency but will show participation of other frequencies (sub-harmonics and super-harmonics) and thus a plot of the amplitude of motion versus the frequency of excitation is no longer meaningful. Results must be obtained in this case for each specific situation.

The dynamic analysis of a machine foundation has to consider on one hand the steady state response to millions of cycles of a harmonic excitation with the frequency of the machine (rotation velocity in cycles per second) and on the other the transient response to starting and stopping conditions, particularly if the natural frequency of the system is smaller than the frequency of the machine. In the first case it is customary to estimate the natural frequency of the system and to try to make it lie outside a range centered at the frequency of the machine in order to avoid resonance or high amplifications. This avoids having to compute the amplitudes of the resulting vibrations, yet this computation, while involving complex quantities, is extremely simple requiring only in most cases the solution of a system of two equations with two unknowns. In any case it is important for this type of analyses to have accurate values of the foundation stiffness particularly at the natural frequency of the system and at the frequency of the machine. The transient analysis requires going from the time domain to the frequency domain and back using Fourier transformation techniques but is more tolerant of small inaccuracies in the frequency variation of the stiffness.

Effects of Local Soil Conditions on Earthquake Motions.

The fact that earthquakes recorded at different sites had very different characteristics (frequency content) depending on the soil properties, as evidenced by seismic motions recorded in the valley of Mexico City, was recognized by Kanai who published in 1957 the first simplified model for soil amplification studies replacing a soil layer by an equivalent single degree of freedom system. This work was complemented by Duke's in 1958 and Murphy's in 1960. Further research was conducted in the late sixties by Donovan and Mathiesen, Seed and Idriss, Roesset and Whitman, and Tsai. Both rigorous analytical solutions based on the Haskell and Thompson transfer matrices for soil layers and discrete models with lumped masses and springs were developed and the convergence of the discrete results to the continuous ones as the number of masses increased was proven. The initial studies considered shear waves propagating vertically through a soil deposit and pointed out the difference between the amplification from a real or hypothetical outcropping of rock to the free surface of the soil and the ratio between the amplitudes of motion (amplification) at the top (free surface) and the bottom of the soil profile (bedrock). The fact that for typical soils with soil stiffness increasing with depth the waves will propagate almost vertically near the surface led to the contention that considering other angles of incidence was unnecessary. Yet solutions for SH waves propagating at arbitrary angles in the underlying rock showed that the amplification from rock outcrop to the free surface of the soil, that would be used to obtain seismic records on soil consistent with a given motion on rock outcrop, are in fact affected by the angle of the waves, even if the incidence is almost normal near the surface in all cases. While the overall shape of the amplification function remains very similar the amplitude of the peaks is substantially reduced as the angle of incidence in the rock increases. The solution for plane wave fronts with arbitrary combinations of SV and P waves was developed very soon after by Jones.

Figure 4 shows typical amplification curves from rock outcrop to the free surface of a homogeneous soil layer for SH waves travelling at various angles of incidence in the underlying rock. It can be seen that the general shape does not change very much (there is a shift in the peak frequency to the left but this is not clearly apparent until the angle with the normal becomes large). The amplitude of the peaks decreases however with increasing angle of incidence. Figure 5 shows the amplification curves for trains of SV and P waves travelling again at various angles of incidence of the P waves in the rock. The angle of incidence of the SV waves has a critical value beyond which P waves would not propagate into the other layer but travel horizontally. It can be seen that the general shape of the amplification functions is again somewhat similar to those for SH waves but there are now some coupling effects between horizontal and vertical motions and the peak at the third natural shear frequency becomes higher than that at the second.
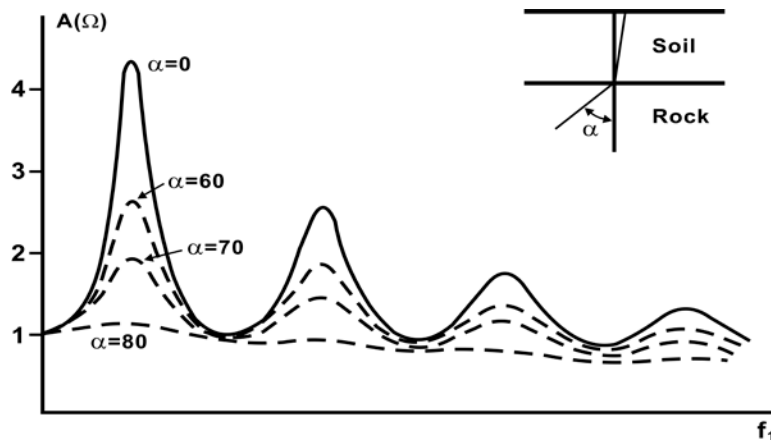

Figure 4.Amplification for SH waves propagating at various angles.

All these solutions assumed linear elastic soil behavior but it was clearly understood that soil is a highly nonlinear material. To account in an approximate way for nonlinear soil behavior Seed and Idriss suggested the use of an iterative linear procedure to define an equivalent linear system. Starting from laboratory curves relating for the soil of interest shear modulus and damping to shear strain and dividing the soil layer into a number of thin sublayers, a linear analysis is conducted obtaining the time history of shear strains at the midpoint of each sublayer. If one were dealing with a harmonic excitation the amplitude of this strain would then be used to obtain from the experimental curves corresponding values of shear modulus and damping for each sublayer. The analysis would then be repeated finding new values of strain, new shear moduli and new damping ratios until the results from two consecutive runs differed by less than a specified tolerance. The first problem with this approach is that an earthquake is not a periodic single frequency (monochromatic) excitation. When dealing with a transient response representing an evolutionary process it is unclear what value of shear strain should be used. After various suggestions using the average of a certain number of peaks it was decided to use the maximum strain multiplied by a reduction or fudge factor taken typically as $2/\pi$. This is clearly an approximation. The procedure tends to converge reasonably fast when the tolerance is not too

small and tends to produce global results such as maximum accelerations atthe free surface of the soil that are reasonable (within 20 % of the values obtained using an actual discrete model with nonlinear springs reproducing the shear stress-shear strain relation for the soil), but larger discrepancies when looking at displacements or deformations. Studies by Constantopoulos showed that in general the procedure, as commonly applied tends to overestimate the damping at high frequencies (filtering out excessively the high frequency components of motion) and underestimate it for low frequencies. Thus although this is a reasonable engineering approximation to account for nonlinear soil behavior it must be realized that an equivalent linear system can never reproduce accurately all the characteristics of the
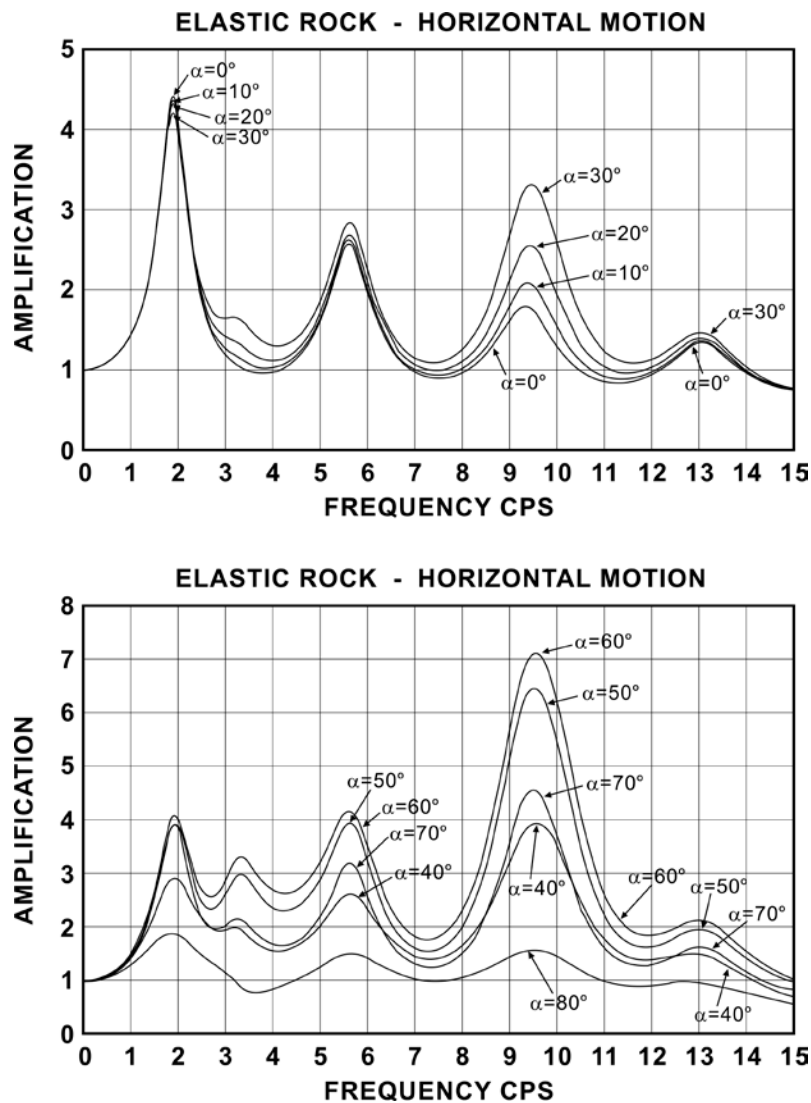

Figure 5. Amplification for trains of SV and P waves at various angles

response of a true nonlinear system and that the discrepancies will increase with increasing level of excitation and nonlinearity. Even if the maximum response is well approximated, the frequency content of the surface motions (and thus the response spectrum of these motions) will be different from what a true nonlinear analysis would predict.

Figure 6 shows the ratio of the response spectra of the motion at the free surface of the soil and the motion at rock outcrop for two different levels of earthquake with a nonlinear solution in the time domain. One can clearly see the shift towards longer periods (smaller frequencies) as the nonlinearity increases and the broadening of the spectrum. The iterative linear analysis would result in each case in an equivalent system with a smaller natural frequency than the original linear elastic soil and an increased value of damping. The general trends would be the same as for the true nonlinear solution but the spectrum for the higher level of motion would be smoother and not as broad.
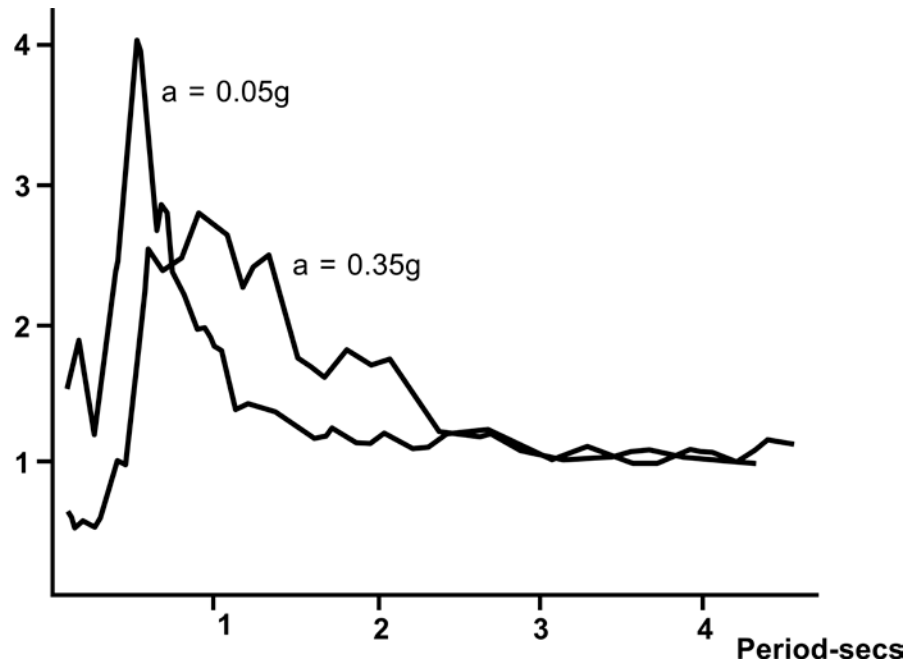

Figure 6. Ratio of Response Spectra for different levels of motion.

Another simplifying assumption of the initial soil amplification studies was the consideration of a horizontally stratified soil deposit where soil properties can vary with depth but not in horizontal planes, leading thus to a one dimensional geometry. There are clearly many situations in which this will not be the case. Not only may the soil layers be inclined but the overall geometry may be clearly two or three dimensional as in the case of narrow valleys or hills. Two dimensional amplification effects in valleys of different shapes (triangular, elliptical, and rectangular) have been investigated by a number of researchers such as Sanchez Sesma. Amplification effects near the base or at the top of hills have also been studied as well as the effects of three dimensional geometries. While the capabilities exist today to find solutions for

all these cases it is difficult to generalize the results of the studies and to obtain approximate expressions due to the large number of parameters involved. In these cases one may have to consider not only the traditional body waves but also surface waves as well as stationary waves trapped in a valley that would explain why two very similar structures on the same soil but at some distance apart from each other (and from the edge of the valley) can experience very different degrees of damage under a particular earthquake (an effect often encountered in real life). A number of authors have pointed out the importance of surface wave amplifications in a number of practical cases and Ruiz and Saragoni showed the importance of free vibrations in the seismic response of the lake zone in Mexico City. All these studies illustrate the limitations of the one dimensional solution for shear waves propagating vertically. Incorporating nonlinear soil behavior in two or three dimensional amplification studies would require on the other hand discrete models with appropriate constitutive equations and a solution in the time domain and there is a scarcity of studies of this kind. The application of the iterative linear approach for these cases raises a number of additional questions in relation to its accuracy. While the results might be qualitatively meaningful they will not be quantitatively reliable particularly for moderate or large excitations.

The use of the one dimensional solution for shear waves vertically propagating through a horizontally stratified soil deposit, combined with the iterative equivalent linearization scheme has been often required in the seismic analyses of special structures, in spite of its many limitations and inaccuracies, in order to obtain site specific design response spectra. In combination with data from motions recorded at a variety of sites this solution has served also to obtain the seismic design coefficients (response spectra), incorporating approximately the effect of soil conditions, proposed or stipulated in a number of codes. The classification of soils in these codes varies widely but it rarely accounts explicitly for the natural frequency of the deposit that is a significant variable. At the same time the codes tend to ignore the fact that the frequency content of the earthquake motions that can be expected at a specific site is not in general (with a few exceptions) a function only of the soil properties but that it depends also on the frequency content, types, relative amplitudes and angles of incidence of the incoming waves, the magnitude of the earthquake, the type of focal mechanism, the distance to the fault, etc. An earthquake does not consist of a train of plane waves but of a combination of many types of waves originating at different points along the fault and arriving at different angles and with different velocities. The motions experienced at two points some distance apart cannot be expected therefore to be identical or to have only a time delay. This obvious consideration has led to the definition of a motion incoherence function. Attempts to obtain this function from actual records ignore again the fact that the incoherence will not be the same for all sites or all earthquakes irrespective of their basic characteristics.

Thus while a great deal of knowledge has been acquired about the dependence of the characteristics of seismic motions on the local soil conditions, while the simple one dimensional theory provides results that are useful and qualitatively reasonable, and while the ensuing code

provisions represent a clear improvement over previous practice, one should keep in mind that there is still much more to be learnt. Additional knowledge would come from our ability to simulate earthquake motions as a function of mechanism, magnitude, distance and topographic conditions in addition to the local soil properties. There is at present a substantial amount of research going on in this area. It should also be noticed that the use of different design spectra for various types of soils is an attempt to account for the effects of the soil on the frequency content and amplitudes of the expected motions but that it does not include any consideration by itself of other possible effects during and after the earthquake such as differential settlements, large deformations, slides, liquefaction or ground failures in general.

Seismic Soil Structure Interaction.

The effect of the flexibility of the foundation/soil system on the seismic response of buildings was addressed by Martel as early as 1940. In the fifties Housner and Merritt and Housner looked again at this problem, using data recorded at and near a building. In the sixties a number of contributions by Sandi, Lycan and Newmark, Monge and Rosenberg, and Hashiba and Whitman appeared in the literature. The main effects of the foundation flexibility, changing the effective natural period and the effective damping of the system, were described by Parmelee using a simple model that has been extensively used since. Parametric studies along these lines were conducted in the early seventies by Sarrazin. By that time it had become accepted that these dynamic soil structure interaction effects would not be generally important for very flexible buildings on rock or very stiff soils, but that they could be significant for very stiff and massive structures such as Nuclear Power Plants. Kausel pointed out the need to consider in the seismic case not only the deformations of the soil due to the inertia forces in the structure (axial forces, base shear and overturning moment), which corresponds exactly to the problem of interest in the design of vibrating machine foundations, but also the effect of a rigid foundation on a train of travelling seismic waves, filtering out high frequency components of the translational motions and introducing rotational motions (rocking and torsion in the general case). To account for these effects in a linear analysis he suggested a three step or substructure approach. Whitman introduced the terms *inertial* and *kinematic* interaction to distinguish these two types of effects. Studies by Luco and Wong, and Morray confirmed the potential importance of kinematic interaction effects particularly for embedded foundations. Although much remained to be done to be able to accurately predict all aspects of seismic soil structure interaction in the real world by the late seventies, after some controversy related to the advantages or limitations of different analysis procedures, the basic phenomena were well known and understood. Even so we have seen in recent years much of these well known aspects being rediscovered with no reference to the original work. In 1985 Wolf's book on *Dynamic Soil Structure Interaction* was published providing a rigorous and comprehensive treatment of the topic with applications both to machine foundations and particularly to the seismic case.

The main consequence of soil-structure interaction is that the motion that will occur at the base of a structure will not be equal to that experienced at the same level in the free field as was traditionally assumed by structural engineers in seismic structural analyses. The differences between these motions are due in part to the scattering of the seismic waves by the foundation (the inability of a stiff foundation to follow the deformations that would occur in the soil) and in part to the deformations and displacements induced in the soil by the inertia forces in the vibrating structure transmitted through the foundation. The first effect is the kinematic interaction, particularly important for embedded foundations. The second is the inertial interaction. In this case instead of finding what would be the motion at the base of the structure in order to conduct a traditional seismic analysis it is normally preferred to analyze a modified system consisting of the structure and the foundation represented by a dynamic stiffness matrix. As pointed out earlier in relation to the problem of vibrating machines for a rigid or very stiff structure this matrix would have at most 6 degrees of freedom; for foundations with 2 planes of symmetry one could uncouple 2 two by two matrices corresponding to horizontal motions and rotations and two independent terms representing the vertical and torsional stiffness; for surface foundations the coupling terms between horizontal translation and rocking are small and could be neglected leading to a diagonal stiffness matrix (or six independent frequency dependent springs and dashpots) whereas for embedded foundations this would require placing the springs at some depth. The effects of the inertial interaction are represented then by the change between the dynamic properties (natural frequencies and damping) of the structure-foundation system and of the structure alone.

Kinematic interaction effects are characterized by a filtering of high frequencies in the translational components of motion and the appearance of rotational (rocking and torsion) components. When subjected to seismic waves travelling at a nonzero angle with respect to the vertical direction even a perfectly symmetric structure would thus be subjected to torsion. In reality both effects take place simultaneously and one should not consider one and ignore the other. Kinematic interaction effects are particularly important for embedded foundations. Their importance will depend on the ratio between the natural frequency of the structure-foundation system and the natural frequency of the embedment layer. It will be very small for low values of this ratio and will become significant as the ratio increases (values larger than 0.5). For stiff, short and wide buildings on soft soils the effect will be generally beneficial whereas for slender structures the base rotation may be detrimental.

Figure 7 shows the effects of kinematic interaction on the horizontal motion of an embedded foundation as the ratio of the amplitude of motion of the foundation to that experienced on the free surface of the soil in the free field. It can be seen that for small frequencies (flexible structures on stiff soils or with little embedment) the effect will be very small but for high frequencies (stiff structures on soft soils with substantial embedment) the reduction in the amplitude of the motions can be considerable. The figure can be approximated as a cosine curve starting at a value of 1 for zero frequency followed by a horizontal line. The transition point

between them occurs at a frequency approximately equal to 0.7 times the natural frequency of the embedment layer. Figure 8 shows the effects of the base rotation measured by the resulting vertical displacement at the edge of the mat. The rotation will be very small for low frequencies but becomes nearly constant in the average (with some fluctuations) for high frequencies. It is important to notice that several popular computer programs for dynamic structural analysis are incapable of accounting for a base rotation and will not reproduce properly soil structure interaction effects in spite of their claims to the contrary.
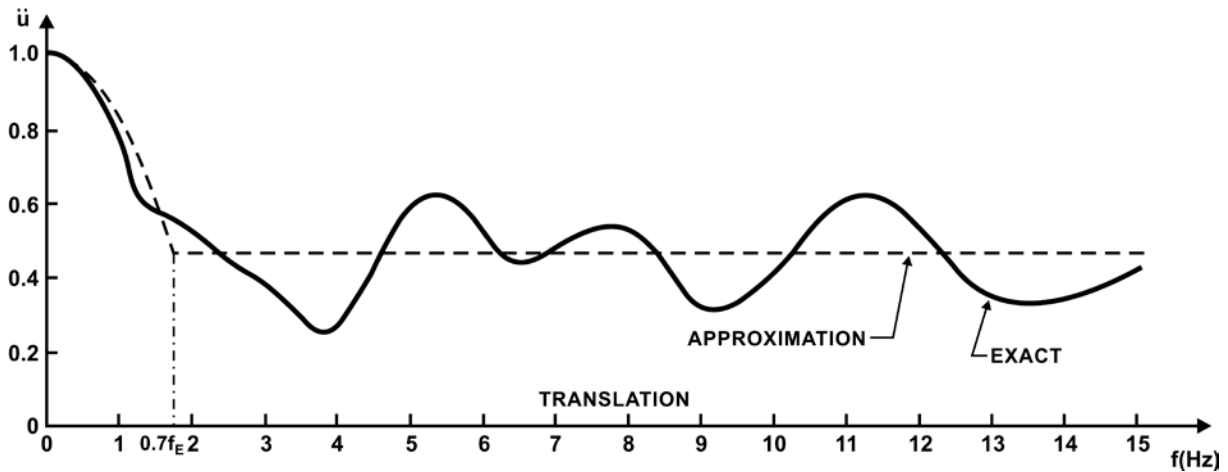
Figure 7. Ratio of translation of embedded mat to that of soil in the free field
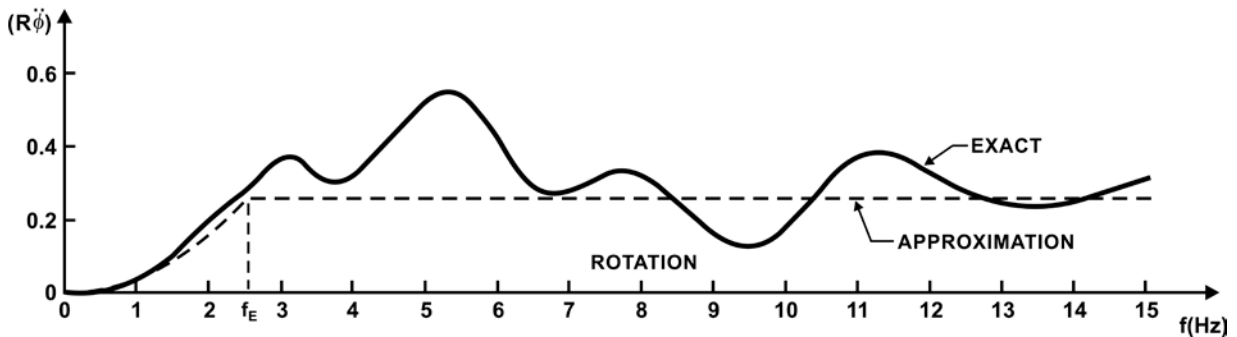
Figure 8 Vertical displacement at edge of embedded mat

Inertial interaction effects are characterized by an increase in the natural period (the structure-foundation system is more flexible than the structure alone) and a change (often an increase) in the effective damping due to radiation of waves away from the foundation. The importance of the change in period will depend on the value of the period of the structure by itself and the frequency content of the seismic motion (including kinematic interaction effects). For any particular earthquake the result may be beneficial or detrimental depending on whether the shift in period leads to a lower or a higher value of the response spectrum. When using smooth design spectra rather than actual motions the effect will be often small. For other types of excitations

(such as wave loads the change in period may be detrimental. The change in effective damping is normally beneficial, particular for short and wide stiff structures, but it could be again detrimental for slender structures because the radiation damping in rocking is much smaller than in translation.

 A number of approximate expressions have been suggested in the literature to estimate the magnitude of inertial interaction effects. Calling T_0 and ω_0 the natural period and natural circular frequency of the structure on a rigid base and T, ω the corresponding quantities including inertial interaction, approximately

$(T/T_0)^2 = (\omega/\omega_0)^2 = \alpha = 1+k/k_z$ for vertical vibrations and

 $(T/T_0)^2 = (\omega/\omega_0)^2 = \beta = 1+k/k_x+kh^2/k_r$ for horizontal vibrations

where k is the equivalent (vertical or horizontal) stiffness of the structure modeled as a single degree of freedom system, h is the height at which the equivalent mass would be placed and k_z, k_x and k_r are the vertical, horizontal and rocking stiffness of the foundation. Calling D_{str} , D_{soil} the internal damping in the structure and the soil, assumed to be of a hysteretic, frequency independent type, and c_z , c_x, and c_r the values of the equivalent vertical, horizontal and rocking dashpots for the foundation and defining

$R_z =(k/k_z)/(1+(\omega.\ c_z/k_z)^2)$ $R_x =(k/k_x)/(1+(\omega.\ c_x/k_x)^2)$ $R_r =(kh^2/k_r)/(1+(\omega.\ c_r/k_r)^2)$

the effective damping at the natural frequency ω is approximately

$D_{zeff} = (D_{str} +D_{soil}\ R_z + 0.5\ \omega\ c_z\ R_z/k_z)/(1+R_z)$ for vertical vibrations and

$D_{xeff} = (D_{str} +D_{soil}\ (R_x+R_r) + 0.5\ \omega\ (c_x\ R_x/k_x+ c_r\ R_r/k_r))/(1+R_x+R_r)$ for the horizontal case

All the above studies have assumed linear elastic behavior. When dealing with moderate or large seismic excitations it will be necessary to account for nonlinear soil behavior. In addition to the nonlinear effects due to the wave passage in the free field, without a structure or foundation, as discussed for the soil amplification problem, one will have to consider the additional strains caused by the vibrations of the structure (the inertial interaction effects). Nonlinear behavior can also be expected to occur in the structure itself for conventional buildings since according to the prevailing design philosophy in most seismic codes buildings are designed to prevent their collapse but allowing inelastic action. A number of studies have been conducted to study interaction effects for nonlinear structures while assuming the soil to remain linearly elastic, which is not a very logical assumption in most cases. Other types of nonlinearities that can be encountered are the separation between the foundation mat and the soil (sliding and uplifting) as studied by Scaletti, or the separation (sliding and gapping) between a pile and the soil near the pile head as accounted for in the P-y curves, and studied by Angelides, Nogami and Novak. It has been customary in soil-structure interaction studies to conduct linear analyses using as soil

properties the reduced values obtained from the iterative linear analyses proposed by Seed and Idriss in the free field soil amplification (wave propagation) studies.

An alternative to the separate consideration of kinematic and inertial effects is to combine them by considering the complete structure-foundation-soil system with a compatible motion specified at the base of the soil deposit. This would require in general a deconvolution of the specified motion at rock outcrop or at the free surface of the soil in order to obtain the compatible base motion, a process that involves a number of questionable approximations, particularly for deep soil deposits. Very often in this case the resulting motion at the free surface for a model of the soil without structure or foundation will not be the same as that originally specified, but will have high frequency components (and at times not so high frequencies) suppressed. Some studies have been conducted with this approach using the iterative linear scheme for the combined soil amplification/soil structure interaction analyses, a discrete model consisting of the soil, the foundation and the structure and determining its dynamic response to a specified base motion at the bottom of the soil. Kim conducted for instance studies of this type for nonlinear single degree of freedom systems supported on pile foundations. While the results of such models can provide some valuable qualitative feeling for the behavior of the system under different levels of excitation their accuracy is questionable. The main objections raised in relation to the validity of the linearization for one dimensional problems are exacerbated when dealing with two and three dimensional states of strain. The consideration of the complete structure-foundation-soil system would be necessary and more logical if one wanted to conduct true nonlinear analyses in the time domain using appropriate nonlinear constitutive models for the soil, but if the design earthquake is specified at the free surface this would require conducting two separate but consistent nonlinear analyses: one for the deconvolution process, the other for the analysis of the complete system.

Once again the main limitations in our present analysis procedures are in the consideration of more general geometries and primarily in the accurate modeling of nonlinear effects.

Soil Characterization by Geophysical Methods.

All the theoretical formulations and computational capabilities developed for the solution of soil dynamics problems are of very little use if one does not know the soil properties in situ accounting for their spatial variability and their variation with levels of strain. Thus the recent emphasis on the determination of the soil properties in the field to obtain first the values of the elastic constants in the linear elastic range, under very low levels of strain, and in the laboratory to develop the appropriate nonlinear constitutive models.

The determination of the elastic properties of soils under low levels of strain using dynamic loads, both in the laboratory and in the field, is based on the principles of wave propagation in elastic media. These principles were established by Poisson, Cauchy and Green in the 1820s and 30s, with important contributions from Stokes in the 1840s. Poisson was the first to identify two

types of waves in an elastic full space (an elastic continuum of infinite dimensions): one associated with compression and dilatation without any shear deformation, and another one with only shear deformation and no volumetric changes. The first one travels faster and is therefore the first one to arrive (primo) being known as the P wave; the shear wave is known as the S wave because it would be the second to arrive. The existence of surface waves when dealing with a half space and a free surface was discovered by Lord Rayleigh towards the end of the 19th century.

Dynamic laboratory tests can apply an impact (short duration Impulse) or a harmonic load to the soil sample. In the first case one measures typically the arrival times of the waves (directly, through phase differences, or through cross-correlation functions) to obtain the wave propagation velocities. Knowing the mass density of the material one can then compute the appropriate elastic modulus (the constrained modulus when dealing with P waves, Young's modulus of elasticity for rod waves, and the shear modulus for S waves as in the case of a torsional excitation). When subjecting the sample to harmonic vibrations at varying frequencies (waiting for each one until a steady state condition has been reached) one obtains a frequency response (or amplification) curve. From this curve one can then determine the frequency at which the maximum amplification occurs (very close to the natural frequency of the specimen if the damping is small) and the amount of damping at this frequency. Since this would be internal, material, damping the values obtained would change with the level of excitation (very small for small amplitude vibrations and increasing as the nonlinear behavior increases). Increasing the level of the excitation would also change the value of the peak frequency from which the corresponding elastic modulus would be derived (secant modulus in the nonlinear range). It is important to notice that under a vertical impulse the first wave to arrive is the P wave with a velocity proportional to the square root of the constrained modulus. This arrival may be hard to detect in some cases. On the other hand when dealing with harmonic steady state excitations depending on the boundary conditions and the load distribution the peak frequency may be associated with the P waves or with the so called rod wave whose velocity is proportional to the square root of Young's modulus. For low values of Poisson's ratio the difference between the P and rod wave velocities is small but it increases with increasing Poisson's ratio. Damping can be obtained from the frequency response curve or from free vibration decay stopping the excitation suddenly once the steady state has been reached, particularly at the resonant frequency.

There are a number of field tests that are commonly used by geophysicists (seismic refraction for instance) that are of limited value to geotechnical engineers because they measure primarily P wave velocities and these are not very meaningful below the water table. The most commonly used methods in geotechnical engineering practice are the Downhole, Uphole and Crosshole tests as well as the Spectral Analysis of Surface Waves (SASW). In all these cases a dynamic load is applied on the surface or at some depth within the soil deposit and velocities or accelerations are measured at receivers placed in other locations. The wave propagation velocities are then determined by a variety of methods. The interpretation of the results in the first three methods is

normally based on simple ray theory assuming a plane train of waves. This provides in general very good results but there may be some exceptions. Clearly the waves propagating as the result of a small (nearly punctual) source within an elastic medium will travel in all directions and consist of different types of waves. As pointed out by Sanchez Salinero even in the simplest case of a full space and an impulse applied at one point, the motions recorded at another point in the direction of the force (longitudinally) will exhibit a main first arrival at the time of the P wave and a second, smaller one, at the time corresponding to the S wave; a point in a line orthogonal to the direction of the applied force (transverse direction) will experience a small motion in the direction of the load starting again at the time of arrival of the P wave and a larger one at the time of arrival of the S wave. Referring to the arrival of the P wave in the first case and that of the S wave in the second as the primary response (the one that would be considered in ray theory) and to the other one as secondary, the latter will decay much faster and essentially disappear at large distances from the source (far field). In the near field however the time of arrival of the S wave may be hard to detect. By the same token the interpretation of the data recorded with the SASW test was based initially on the assumption of a pure Rayleigh wave (the first Rayleigh mode initially and other modes later). This would correspond to a two dimensional solution but the actual situation is three dimensional, particularly in the near field.

Figure 9 (after Sanchez Salinero) shows the motions in the horizontal direction that would be recorded at a point due to an impulse applied at another in the same direction. Ray theory would consider only a P wave. One can clearly see the arrival of the P wave and then the secondary motion starting at the time of arrival of the S wave. Figure 10 shows the corresponding results for the vertical motion due to a vertical excitation. The primary motion would be associated with SV waves but the secondary motion starts in fact at the time of arrival of the P wave. These are near field effects that become negligible for large distances between the source and the receiver.
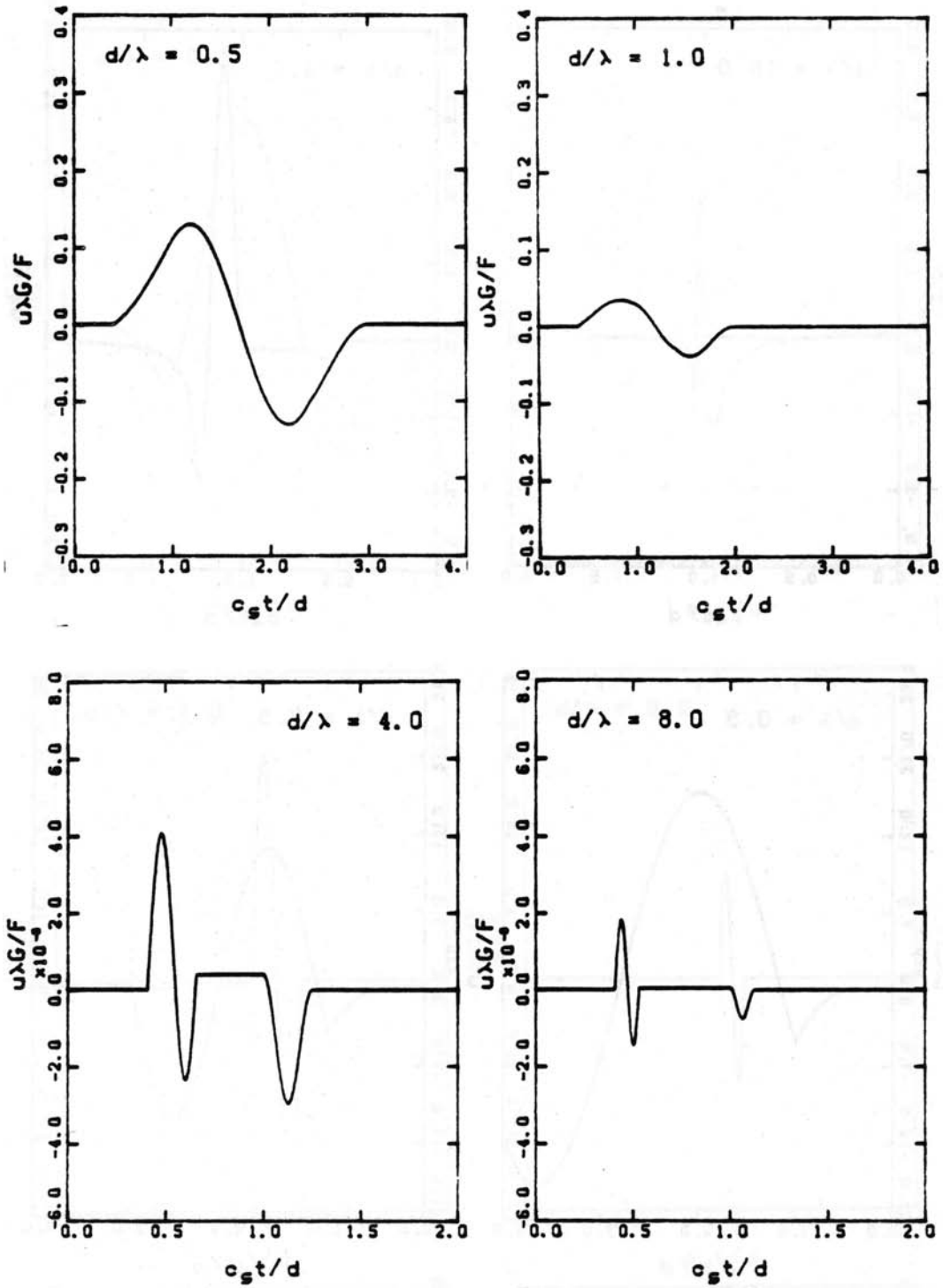
Figure 9. Longitudinal displacements at various distances due to longitudinal pulse
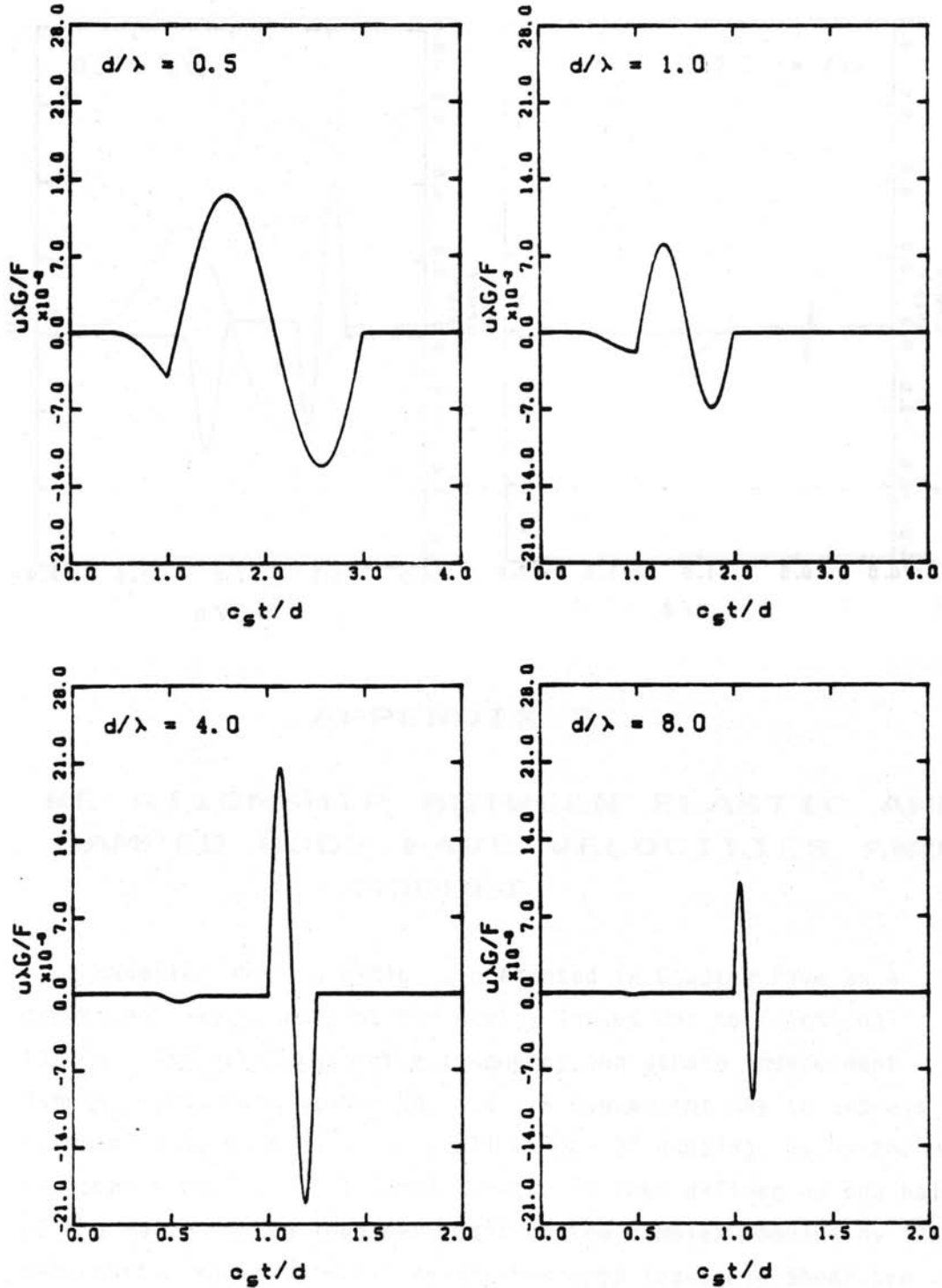
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Figure 10. Transverse displacement at various distances due to transverse pulse

Figure 11 (after Foinquinos) shows the theoretical amplitude of the vertical displacements on the surface of an elastic half space due to a vertical point load for different values of Poisson's ratio as well as the far field solutions. The ordinates are the amplitudes of the displacements
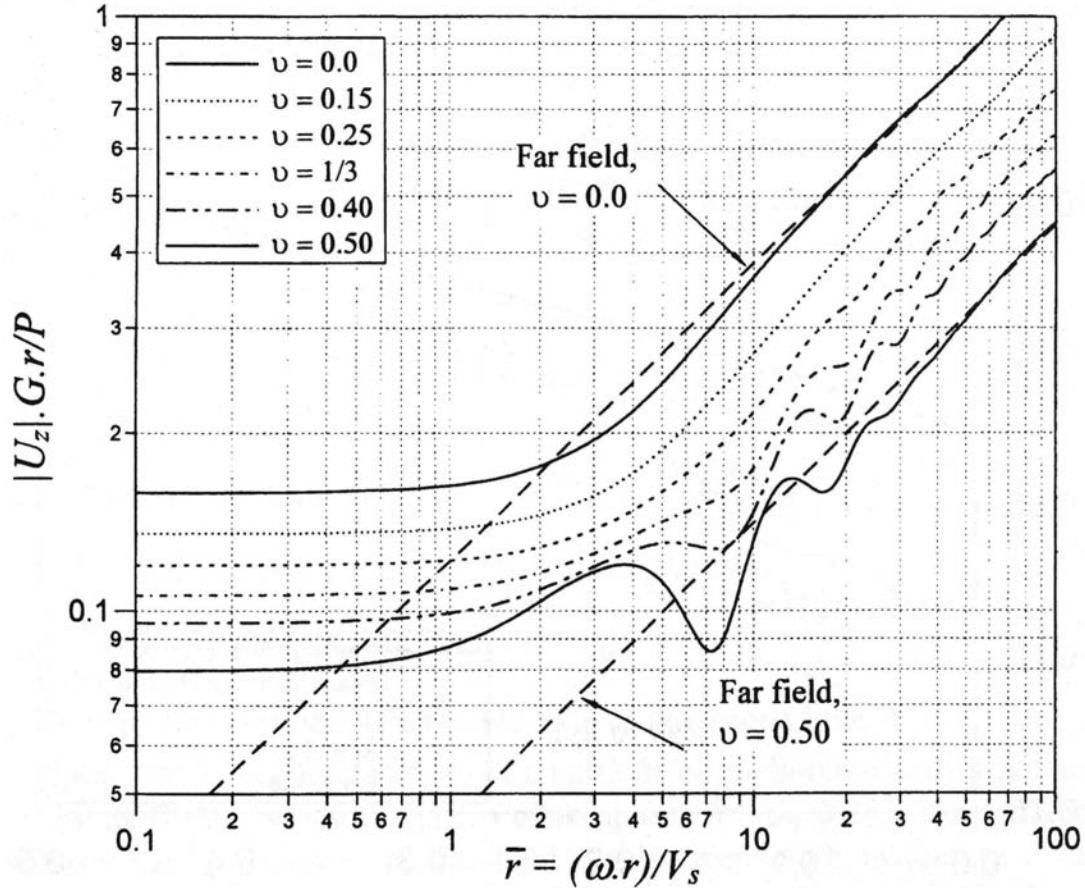
Figure 11. Vertical displacements due to vertical load on the surface of a half space

multiplied by the distance in dimensionless form; the abscissas are dimensionless distances. The figure is in logarithmic coordinates. A horizontal line in this figure (as corresponding to small values of the distance or the near field) indicates a displacement amplitude inversely proportional to the distance; a sloping line with a slope of 0.5 (corresponding to large distances or the far field) indicates a decay with the square root of the distance. It can be seen that even for large distances the amplitudes fluctuate around the pure Rayleigh wave solution with large values of Poisson's ratio. This effect is further illustrated in figure 12 (also after Foinquinos) where the phase velocities that would be recorded at different distances normalized by the pure Rayleigh wave velocity are shown versus distance for different values of Poisson's ratio. The assumption of a pure Rayleigh wave may therefore lead to inaccuracies in the estimation of the soil properties.

A rigorous interpretation of the experimental data requires in all cases the solution of Lamb's problem finding the displacements, velocities or accelerations induced at any point within a layered medium by an impulse applied at another point. Lamb formulated this problem in 1904 for the case of a harmonic load applied on the surface or within an elastic, homogeneous and
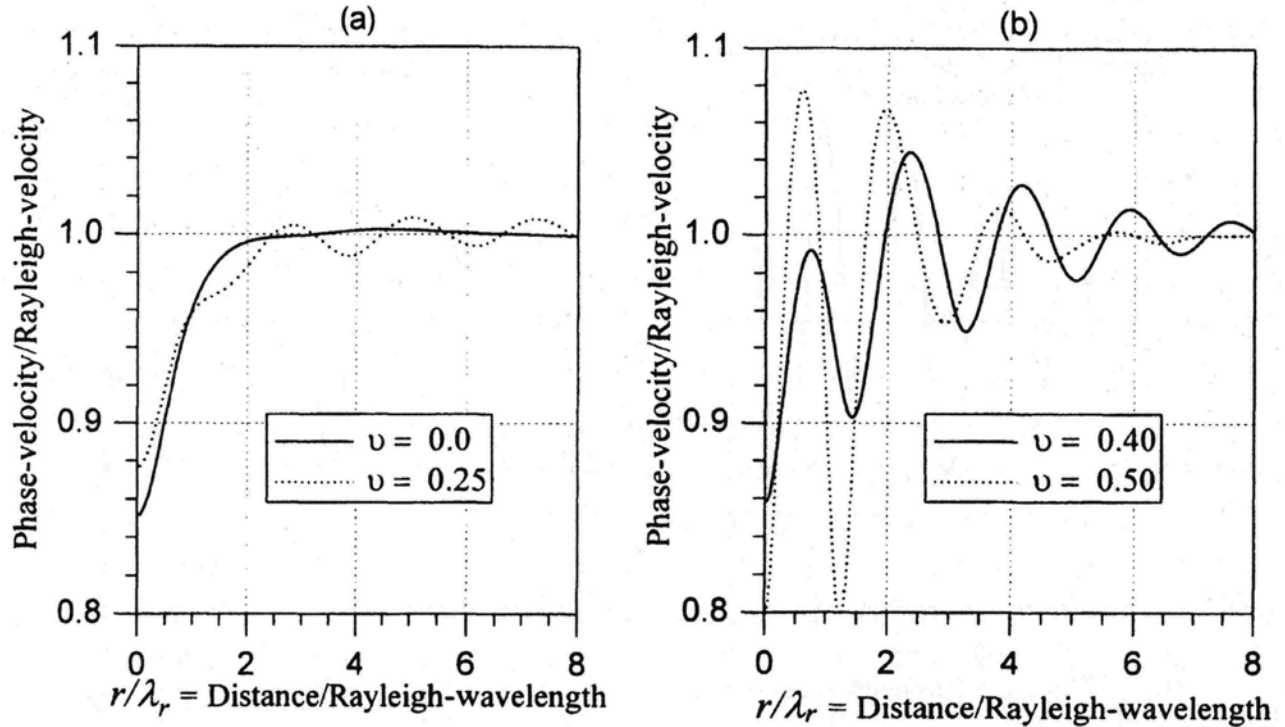
Figure 12. Phase velocities at various distances along surface of a half space

isotropic half space. Expanding in asymptotic form the integrals appearing in the solution he was able to obtain results for the far field (at large distances from the source). Pekeris in 1955 and Mooney in 1974 extended this solution; Kausel in 1981 presented in explicit form the Green's (or influence) functions for layered soils both in time and frequency domains and Luco and Apsel published also in 1983 solutions for a layered medium.

In the downhole method the source is placed on the surface and the motions and wave arrivals are recorded at various depths along a borehole. In the uphole test the source is placed at various depths and the motion recorded at the surface. In both cases one obtains estimates of the wave propagation velocities at various depths at a particular location (the position of the borehole). In the crosshole test two or three boreholes are used. The source is placed at various depths in one of them and the motions are recorded by sensors located at the same depth in the others. One obtains then average velocities at each depth over the distance between boreholes. The assumption is that the waves travel in a straight line between the source and the receivers but this would not be the case if one had a thin soft layer next to a much stiffer one. In a variation of this method one can place geophones at all the desired depths and measure the resulting motions for each position of the source in order to perform tomography. The assumption is again that the waves are travelling in straight lines (ray theory) but Liao has shown that the true situation is more complicated. Tomography has been used to try to identify buried cavities or inclusions or at least the existence of anomalies between the boreholes.

The SASW method is a variation of the Rayleigh wave method developed primarily by Stokoe. In the original method a harmonic excitation was applied on the free surface and the receivers were moved along the surface until the motions recorded were 180 degrees out of phase (half a wavelength). Repeating the process for different frequencies one could obtain a dispersion curve relating wavelength to frequency, phase velocity to frequency, or phase velocity to wavelength. It was then assumed that the phase velocity calculated was the Rayleigh wave velocity. For an elastic half space and the far field, Lamb's solution would predict a constant value of the phase velocity (no dispersion). This was a powerful but laborious and time consuming method. In the SASW method the excitation is a transient impulse (or in reality a series of transient impulses with different durations). For each test with a given impulse duration the motions are recorded at two receivers placed on the surface (one can use more than two receivers if so desired).The time records of the motions at the two receivers are converted automatically to the frequency domain through a spectral analyzer and their phase difference is computed as a function of frequency using the cross spectrum. The phase velocity can then be obtained as a simple function of the phase difference and the distance between receivers. This provides the dispersion curve over an appropriate range of frequencies for each impulse duration and distance between receivers. The results of the tests for different durations are superimposed overlapping each other and a smooth average curve is fitted through the results. The soil properties are estimated from this experimental dispersion curve starting with the highest frequency, corresponding to the smallest wavelength. Because the amplitude of the Rayleigh wave decays exponentially with depth the corresponding phase velocity would represent the value for a small soil layer near the surface. The assumption is made initially that this value applies to a layer with a thickness equal to a fraction of the wavelength (typically between one third and one). Knowing the properties of the top layer one looks then for that of an underlying half space so that the combination of the layer and the half space agree with the results for the next (lower) frequency. One assumes again a total depth equal to a fraction of the wavelength and proceeds to find a half space below the layers with thickness and properties already known that would match the experimental data for the next frequency. Once the process is finished for all frequencies it is necessary to adjust thicknesses and properties so as to match the dispersion curve over the complete range of frequencies since at any given stage the new layering, by opposition to a uniform underlying half space, will introduce deviations in the results for the previous, higher, frequencies. This iterative scheme, adjusting the profile to obtain an optimum match, is hard to automate and requires some experience from the user. Figure 13 shows the soil profile resulting from the first estimates and after refinement, the original dispersion curve and those obtained resulting for the original estimates and the refined profile in the case of a simple soil profile with smooth variation of the stiffness with depth. Results are shown for a solution assuming a plane wave front (2 D) and for the more rigorous 3D solution. In this case both solutions give very close results with a small difference at the peak that occurs around a wavelength of about 3.7 m. Figure 14 shows the corresponding results for a more difficult condition with an initial increase in stiffness followed

by a substantial decrease. In this case the differences in the final dispersion curves obtained by the two approaches are more significant.

An important practical point is the selection of an appropriate receiver spacing for each range of frequencies (pulse duration) if one is going to back-figure the soil properties assuming a plane train of surface waves. Figure 15 shows the phase velocities resulting for different spacing of the two receivers. An optimum result is obtained when the receivers are placed at two and four wavelengths from the source. Since one is not dealing with a single frequency it is common to define the valid range of frequencies so that the distance between receivers is between one and two wavelengths. Distances less than half a wavelength will give rise to important near field effects. One uses then short duration (high frequency) impulses, low amplitudes and short distances between the receivers to estimate the soil properties near the surface. As the depth at which the properties are desired increases one must use longer pulse durations (lower frequencies), higher amplitudes, and larger distances. This implies that the measured soil properties are average ones over different areas for each impulse (the distance between receivers). For large depths one may have to use large amplitude pulses and the soil behavior may no longer be linear near the surface. As the depth increases the resolution of the inversion procedure deteriorates and it is very hard to detect a thin layer of soil with very different properties from the adjacent ones when its depth is much larger than its thickness. The SASW method can also be used to detect anomalies within the soil such as cavities (tunnels, old mines) or inclusions, between receivers. This can be easily achieved when the dimensions of the anomaly are large in relation to the depth at which it is located but becomes much harder as the ratio of the dimensions to the depth decreases. In those cases other techniques such as seismic reflection may have an advantage.

Suddhiprakarn studied the effect of sets of inclusions with different arrangements on wave propagation through an elastic medium using a Boundary Element formulation. He looked at the variations in both the amplitudes of the recorded motions and the times of arrival of the waves for different sizes of the inclusions relative to the wavelength and different contrasts in material properties between the inclusions and the surrounding medium. He compared also the 3D solutions to 1D, ray theory, predictions, showing that this simplified approach yields the same general trends but tends to overestimate the effects of the inclusions, predicting larger variations than should be expected. Nogueira and Tassoulas looked at the use of the SASW method to identify buried cavities studying the effects of cavities on the propagation of surface waves.
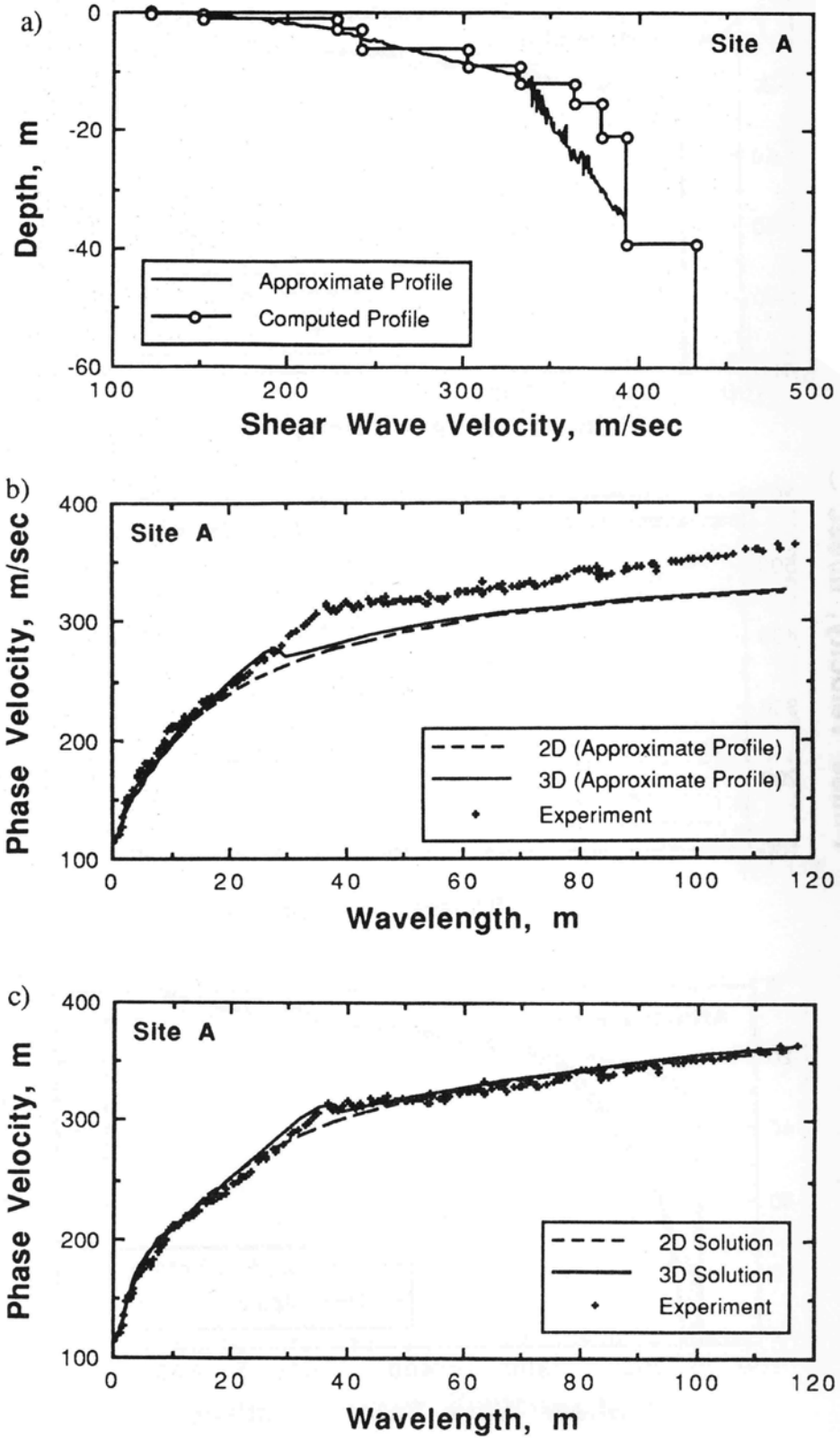
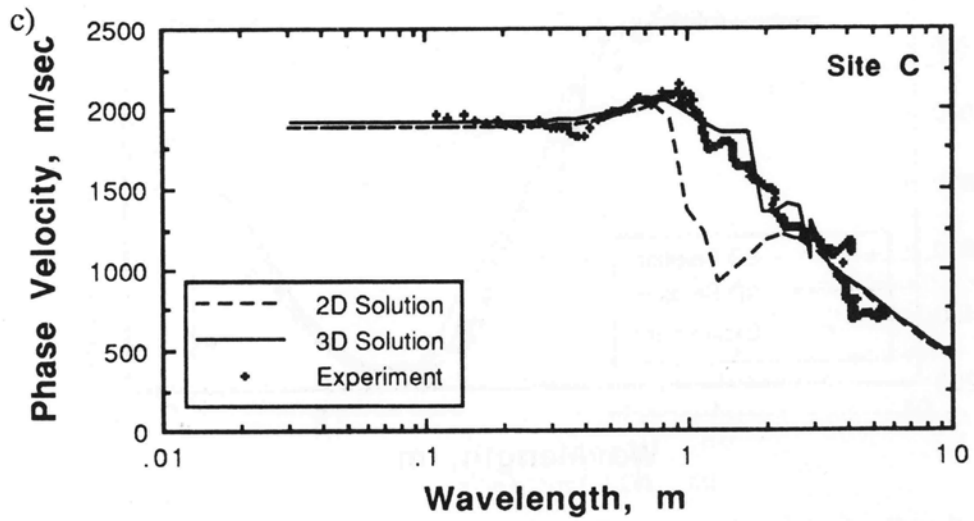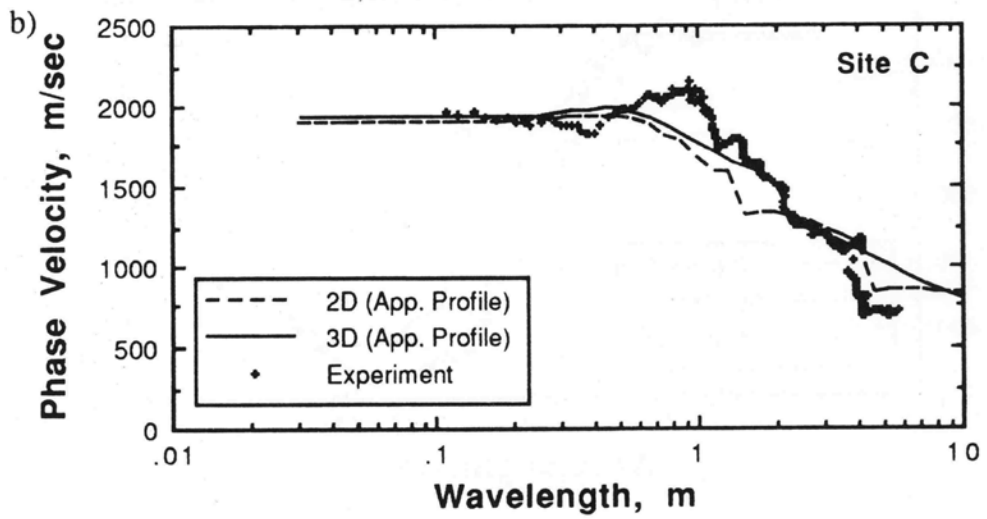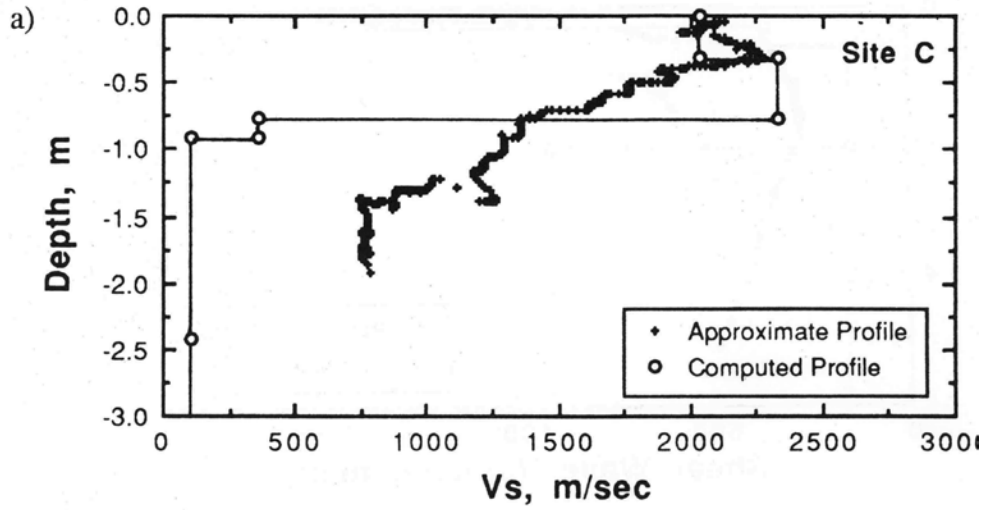Figure 13. Site A. Initial and refined profiles and their dispersion curves

Figure 14. Site C. Initial and refined profiles and their dispersion curves
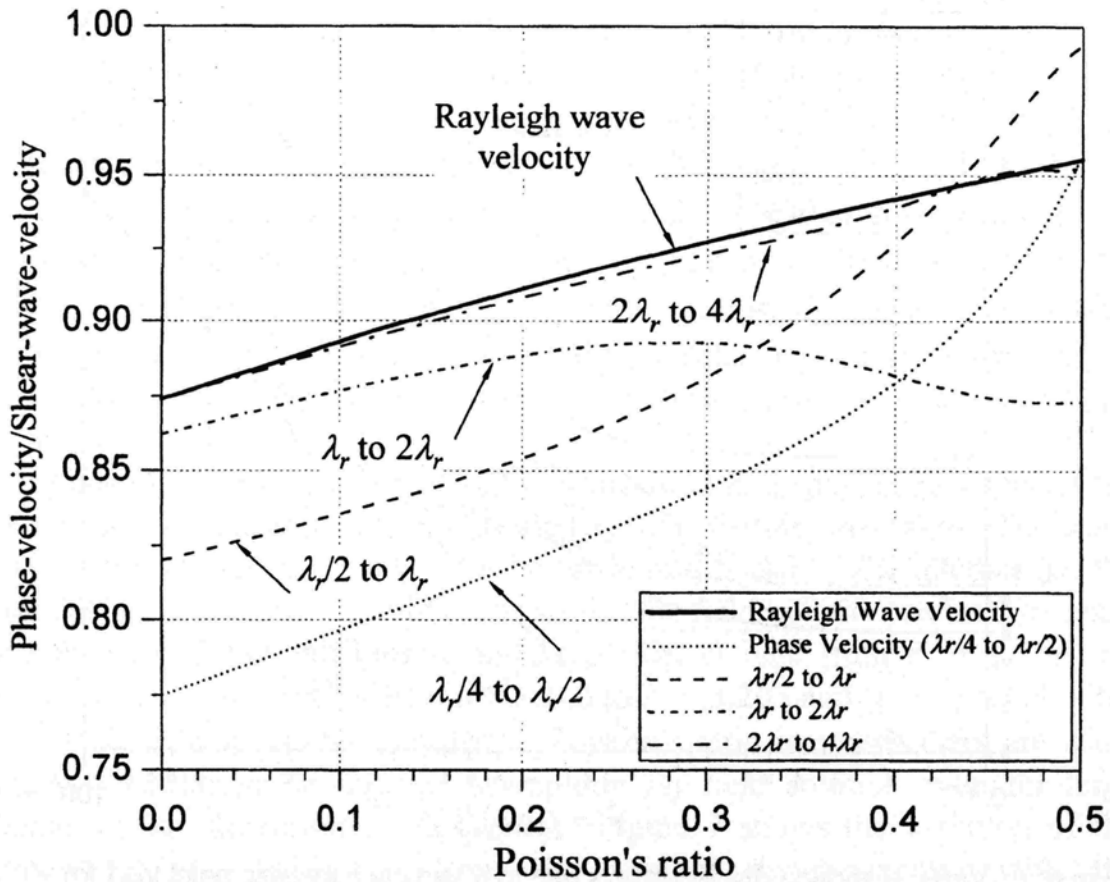
Figure 15. Effect of distance between receivers on phase velocity

Dynamic non-destructive testing techniques have also been extensively used to evaluate the structural integrity and capacity of highway and airfield pavements. These techniques can be grouped into two general categories: wave propagation tests and deflection basin tests. The first group includes the SASW above discussed. Deflection based tests are those in which maximum deflections are recorded along the surface of a pavement subjected to a steady state harmonic load (as with the Dynaflect and the Road Rater) or to a transient impulse (as with the Falling Weight Deflectometer). At present the interpretation of the deflection basin, in order to backfigure the elastic properties of the pavement layers (pavement, base, subbase), is normally performed with static analyses (assuming that the maximum deflections occur at the same time at all the receivers in spite of the fact that the difference in the time of the peak deflections is clearly apparent from the time records). It is also commonly assumed that the subgrade extends to infinity. This approach neglects the dynamic nature of the tests and the fact that the soil will be underlain at some depth by much stiffer rock like material (this will affect the results if the depth to bedrock is small). The effect of the depth to bedrock on the deflection basins obtained with the Dynaflect and the Falling Weight Deflectometer (FWD) were studied by Chang and Kang.

Among these tests the Falling Weight Deflectometer is the test that has seen most widespread use, in part due to its ability to impose loads similar in magnitude to those due to truck traffic. The FWD consists of a drop weight mounted on a vertical shaft and housed in a trailer that can be towed by most conventional vehicles. The drop weight is hydraulically lifted to predetermined heights ranging from 5 to 50 cm and dropped onto a 30 cm diameter plate resting on a 5.6 mm thick rubber pad. The resulting force is an impulse with duration of approximately 30 msec. and a peak magnitude ranging from 9000 to 90000 N, depending on the weight and the drop height. The applied force is measured by a load cell and the resulting deflections by a set of vertical velocity transducers. The values of the deflections obtained with the FWD may be significantly affected by the position of the source and receivers with respect to the edge of the pavement, particularly when they are close to it, as discussed by Kang. At the same time when applying the larger weights there may be some nonlinear soil behavior that is usually neglected, as pointed out by Chang. An excellent study of the dynamic characteristics of the deflection basins obtained with the DFW (using a version developed in Canada) was recently conducted by Grenier.

Moving Loads.

The potential problems for bridges due to moving loads were already identified in the middle of the nineteenth century. It was only much later however that these problems were extensively studied, showing the existence of a critical velocity that could give rise to resonance phenomena. These studies showed also that in order to simulate properly the dynamic response of a bridge to moving traffic it was necessary to have a realistic model of the vehicles as sets of masses, springs and dashpots, and to reproduce properly the geometry of the access to the bridge, accounting for potential differences in level or gaps that would give rise to impact loads as the vehicle entered the bridge.

The study of vibrations caused by traffic and transmitted through the soil is a more recent endeavor. It has become important due to the construction of subways or underground trains that may be traveling very close to the foundations of existing buildings giving rise to both vibrations that can endanger sensitive equipment (in hospitals for instance) and noise. The problem can become more serious when dealing with high speed trains.

The first steam engine freight train operated in 1825 between the coal mine at Darlington and Stockton. The first passenger train was established in 1830 running between Manchester and Liverpool. In the early stages the rail was designed on the basis of empirical relations but in 1867 Emil Winkler published his book *Die Lehre von Elasticität und Festigkeit* where he developed the analytical solution for a beam on an elastic foundation, a model that has been extensively used since for a large number of different applications (beams, plates, piles). For the static case, calling E the modulus of elasticity of the beam, I the moment of inertia of its cross section and k the elastic constant of the foundation (the ballast coefficient multiplied by the width of the beam) the displacement is given by

$$v(x) = \frac{p}{8EI\lambda^3} e^{-\lambda x}(\cos \lambda x + sen\lambda x) \quad \text{for x>0}$$

and $\qquad v(x) = \frac{p}{8EI\lambda^3} e^{\lambda x}(\cos \lambda x - sen\lambda x) \quad \text{for x<0}$

with $\qquad\qquad\qquad\qquad \lambda^4 = \frac{k}{4EI}$

For the dynamic case with a moving load of constant magnitude traveling at a velocity V the displacement at a point with an abscissa x at a time t can be expressed in terms of a variable

$$z=x-Vt$$

There is then a critical velocity V_{cri}

$$V_{cri}^2 = \frac{2\sqrt{kEI}}{m}$$

Calling $\quad \alpha = V/V_{cri} \qquad \lambda^4 = \frac{k}{4EI}\left(1+\alpha^2\right)^2 \qquad \mu^4 = \frac{k}{4EI}\left(1-\alpha^2\right)^2$

the displacement for V< V_{cri} is

$$v(z) = \frac{P}{4\sqrt{kEI}} e^{-\mu z}\left(\frac{\cos \lambda z}{\mu} + \frac{sen\lambda z}{\lambda}\right) \quad \text{for } z \ge 0$$

and $\qquad v(z) = \frac{P}{4\sqrt{kEI}} e^{\mu z}\left(\frac{\cos \lambda z}{\mu} - \frac{sen\lambda z}{\lambda}\right) \quad \text{for } z \le 0$

For z=0 the maximum displacement is

$$v(0) = \frac{P}{4\sqrt{kEI}}\left(\frac{1}{\mu}\right) = \frac{P}{2\sqrt{2}\left(EI\right)^{.25} k^{.75}\sqrt{1-\alpha^2}}$$

For a given abscissa x this expression gives the variation of the displacement at that point with time, as illustrated in figure 16. For a given time t the same expression gives the value of the displacement along the rail with z=0 corresponding to an abscissa x=Vt as the position of the load), as illustrated in figure 17. In both cases the velocity is much smaller than the critical velocity, although the results change little until the velocity approaches the critical value. The maximum occurs then at a time t=x/V corresponding to the load passing over the point. Figure 18 shows on the other hand the deflected shape of the rail as the velocity approaches V_{cri}. It can

be seen that the amplitudes on both sides increase relative to the maximum value at the point under the load and that there are more significant oscillations. It should be noticed that these figures have different vertical scales. The amplitudes of the displacements are the same in figures 16 and 17 but they would be noticeably larger in figure 18.

These expressions have been used for many years and continue to be used today with the value of k being that from a static load application. When using the static value of the ballast coefficient the resulting critical velocity is very large. Considering on the other hand a uniform harmonic line load on the surface of a soil deposit, computing the displacement as a function of the frequency, and defining the stiffness k of the foundation as the value of the displacement divided by the amplitude of the load, one would find that k depends on frequency and decreases with increasing frequency in a way similar to the vertical stiffness of a foundation, becoming 0 at the natural frequency of the soil layer if there is no internal damping. This is illustrated in figure 19 for a particular soil deposit. When dealing with a moving load and a rail over a soil deposit of finite thickness the effective value of k depends on the velocity of the load due to inertia effects in the soil and the critical velocity can be much smaller than the one predicted with the static k, depending on the soil properties.

 S. M. Kim conducted in 1996 a number of studies on moving loads (both constant magnitude and harmonic loads) in connection with the design of a Dynamic Rolling Weight Deflectometer (DRWD) for pavement testing. He obtained and compared solutions for a beam on an elastic foundation, a plate on elastic foundation, and an actual soil deposit under the ballast with an accurate soil dynamics model (accounting even for a load distributed over a small (tire print) area instead of a point load. He discussed the effects of velocities larger than the critical and he showed that for values of the velocity well below the critical, as was the case for the DRWD) all solutions yielded very similar results. The effects of high speed trains on the ballast accounting for the soil are being investigated experimentally in Europe at a facility of the Geotechnical Laboratory of the CEDEX in Madrid.
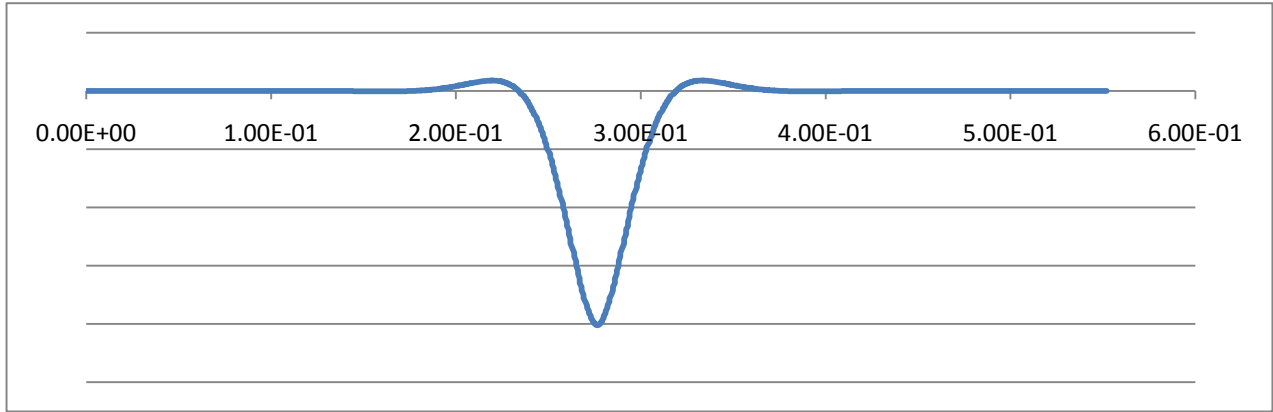
Figure 16 Time variation of deflection at a point due to moving load with V<< V_{cri}
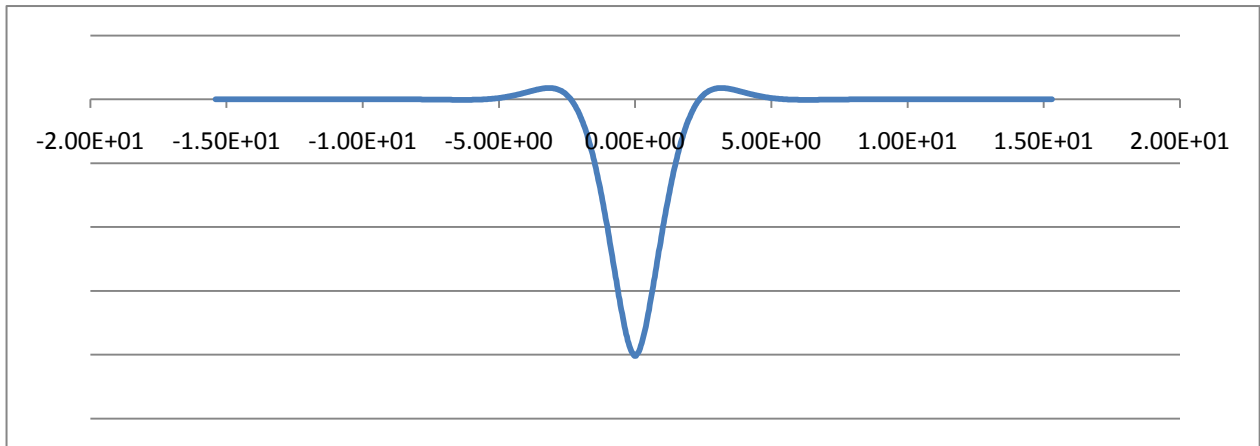

Figure 17. Variation of Displacement with distance to point of application of load. V<< V_{cri}
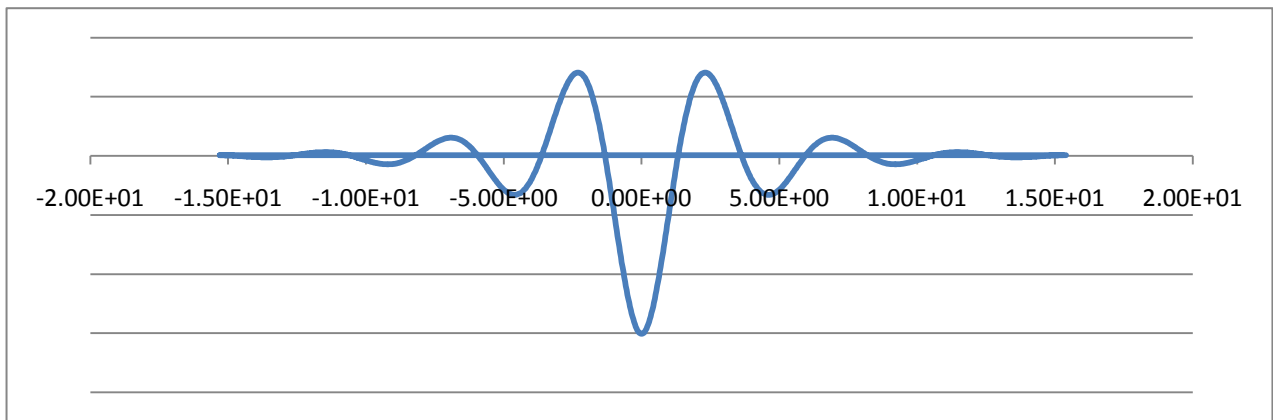

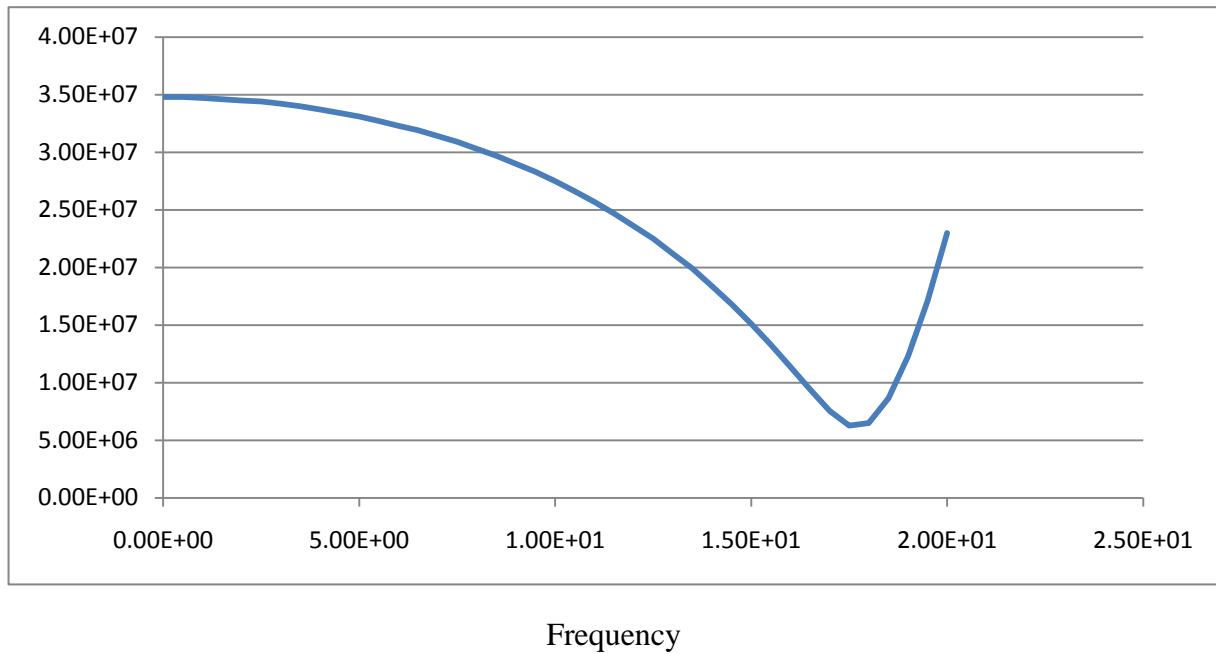Figure 18. Deflection curve for velocity close to the critical velocity

Frequency

Figure 19. Variation of stiffness k of elastic foundation with frequency

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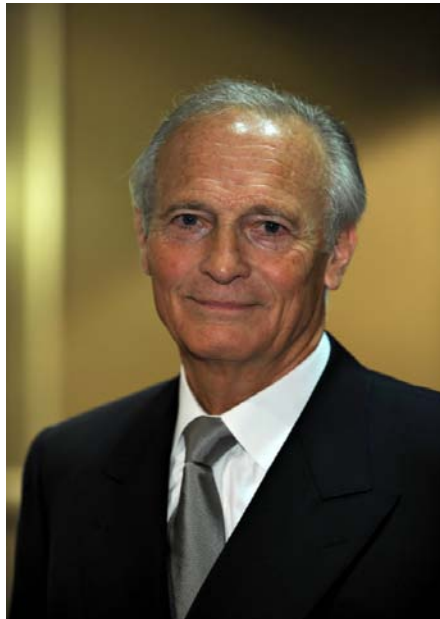
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Jean –Pierre Giroud

The 2008 Terzaghi Lecture was first presented at the GeoCongress of the American Society of Civil Engineers (ASCE). The lecture presented at Texas A&M will be an updated and expanded version of the original lecture. This lecture presents a summary of more than 30 years of work on geotextile and granular filters by the author. A rational approach to the development of filter criteria is presented in a lively manner using animated slides. In particular, the author demonstrates that, while two criteria are needed for granular filters, four criteria are needed for geotextile filters. The author also demonstrates that, while the traditional permeability criterion for granular filters is adequate, the retention criterion for granular filters could and should be improved by adapting some of the features of the retention criterion for geotextile filters. The application of the filter criteria is illustrated and discussed step by step using the case history of the design, construction and performance monitoring of a geotextile filter in a dam constructed in 1970 and still in service.

Dr. Giroud, a former professor of geotechnical engineering, is a consulting engineer under JP GIROUD, INC., and Chairman Emeritus and founder of Geosyntec Consultants, a large consulting company. Dr. Giroud is chairman of the editorial board of *Geosynthetics International* and past president of the International Geosynthetics Society (the IGS). He coined the terms "geotextile" and "geomembrane" in 1977 and has authored over 350 publications. He has developed many of the design methods used in geosynthetics engineering (in particular for leakage through liners, liner stability, unpaved roads, and filters) and has originated a number of geosynthetics applications, in particular for landfills, liquid impoundments, and dams. In 1994, the

IGS named its highest award "The Giroud Lecture", "in recognition of the invaluable contributions of Dr. J.P. Giroud to the technical advancement of the geosynthetics discipline". A Giroud Lecture is presented at the opening of each International Conference on Geosynthetics by a lecturer selected by the IGS Council. In 2002, Dr. Giroud became Honorary Member of the IGS with the citation "Dr. Giroud is truly the father of the International Geosynthetics Society and the geosynthetics discipline". In 2005, Dr. Giroud has been awarded the status of "hero" of the Geo-Institute of the American Society of Civil Engineers (ASCE) and has delivered the prestigious Vienna Terzaghi Lecture in Austria. In 2005-2006 he presented the Mercer Lectures, a prestigious lecture series endorsed jointly by the IGS and the International Society for Soil Mechanics and Geotechnical Engineering. In 2007, J.P. Giroud became Doctor Honoris Causa of the University of Bucharest (Romania) and, in 2008, he delivered the prestigious Terzaghi Lecture of the ASCE. In 2009, J.P. Giroud has been elected member of the US National Academy of Engineering. In 39 years, Dr. Giroud has worked more than 100,000 hours on geosynthetics.

The Terzaghi Lecture: Criteria for geotextile and granular filters

by J.P. Giroud

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Spencer J. Buchanan '26 Chair in Civil Engineering

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