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Specification of control motions for embedded foundations

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ABSTRACT: The location where the control motion is to be specified for the seismic analysis of structures with embedded foundations has been a subject of considerable controversy in recent years, particularly in relation to the design of nuclear power plants. This controversy may be due in part to confusion between the motions that would occur at the base of a rigid and massless embedded foundation (the result of kinematic interaction or wave scattering effects) and those that would take place at the same level in the free field, computed from soil amplification studies. In this paper the theoretical basis for the determination of compatible motions of a massless embedded foundation due to a specified train of seismic waves as well as some experimental data are presented.

1 INTRODUCTION

Seismic analyses of structures were carried out at one time assuming that the design earthquake was the motion that occurred directly at the base of the structure. It has long been recognized, however, that local soil conditions have important effects on the characteristics of the earthquake motions to which a structure may be subjected. These effects are normally studied in three separate phases:

- the amplitude and the frequency content of the seismic motion at a point A (see figure 1) on the free surface or a point B at any level within a soil deposit, before any structure is built, are functions of the types of waves and the soil properties in the linear elastic and the inelastic ranges. This effect is commonly known as soil amplification although the name may be misleading, since there is in fact amplification over certain ranges of frequencies and deamplification over others. The determination of the motions (displacements and accelerations) and stresses at any point within a soil mass due to a specified train of surface or body waves can be easily performed analytically for a horizontally layered soil deposit (properties may vary with depth but remain constant in horizontal planes) assuming linear elastic behavior (or approximating nonlinear behavior through iterative linear analyses). The solution of this problem is still possible, although more expensive, for two or three dimensional geometries of the deposit assuming again linear behavior. It is much more difficult with a true nonlinear analysis.
- the motion of a massless foundation, before any structure is built or if the structure had no mass, would be different from that recorded at the free surface or at any other point within the soil deposit. The differences will consist in general of a filtering of the translational motions (a decrease of their amplitude in the high frequency range) and the occurrence of rotational components of motion. The only case where this effect would not be present is for a surface foundation and vertically propagating seismic waves. For an embedded foundation, of particular interest here, the motions would be different at every point along the interface between the foundation and the surrounding soil. For a rigid foundation, however, they can be expressed in terms of three translational and three rotational components of motion at the centroid of the base contact area (point C in figure 1).
- once the structure is built, or when its mass is taken into account, the

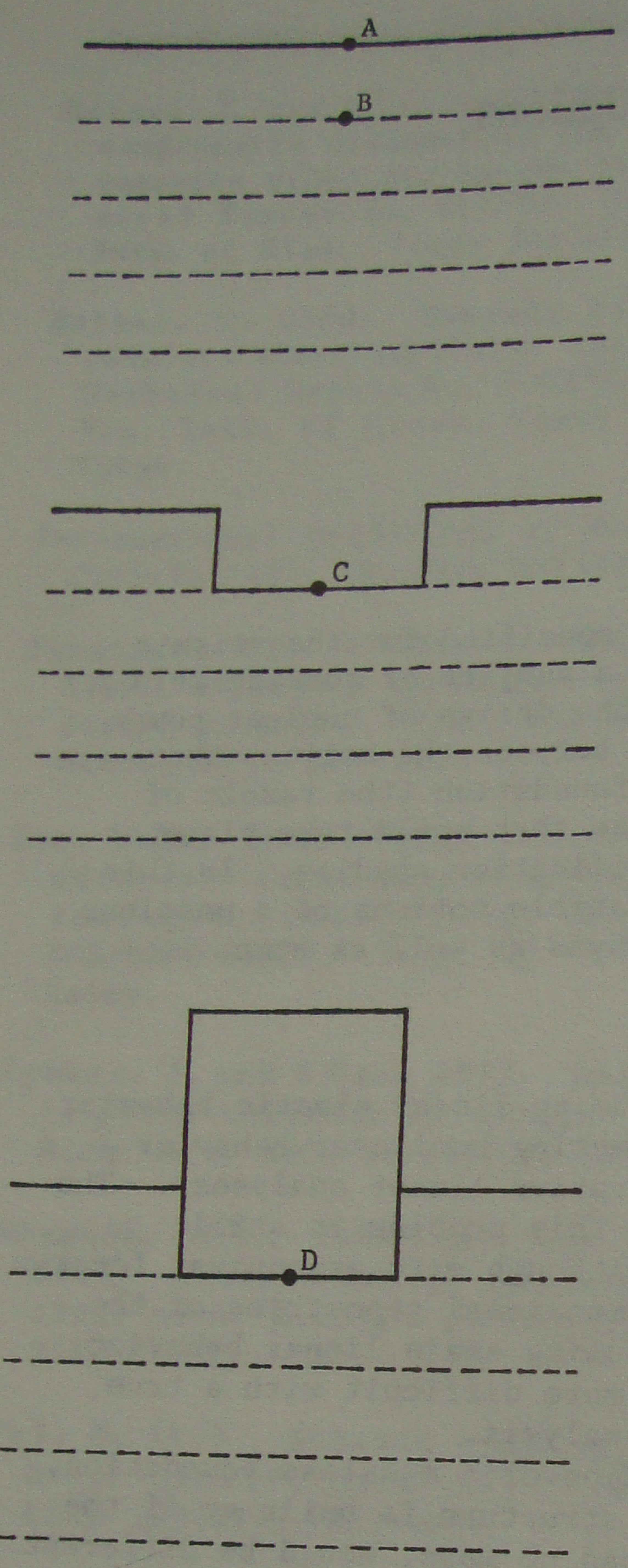

Fig.1 Definition of control points.

inertia forces generated by the structural vibrations will give rise to stresses along the interface between the foundation and the soil which will result in additional deformations and alter the foundation motions. For a rigid foundation these stresses can be expressed in terms of an axial force, two base shears, a torsional moment and two overturning moments at the centroid of the base contact area (point D). The accelerations of this point will thus be different from those that a massless foundation would experience by itself (point C), the motions recorded at the free surface (point A) or at a point at the foundation level in the free field (point B).

These last two effects are commonly known as soil structure interaction. The first one is called sometimes kinematic interaction (function of the geometry of the foundation and the type of seismic waves propagating through the soil), while the second is referred to as inertial interaction (caused by the inertia forces in the structure). It appears that in some cases, however, the term soil structure interaction is used only in reference to the second phenomenon. The relative importance of inertial versus kinematic interaction effects depends on the frequency characteristics of the earthquake as well as the structure's, foundation's and soils' properties. In general inertial interaction will predominate for short, stiff buildings on surface or shallow foundations and soft soils, while kinematic effects will be more important for deeply embedded foundations (and relatively more so for flexible buildings and stiffer soils).

While this division into three separate phases is convenient from a conceptual point of view and to check and interpret the results of seismic soil structure interaction analyses, it is important to remember that the effects must take place simultaneously and are very closely related. If separate analyses are performed for two or all three of these effects, consistent assumptions should be used to obtain results which are physically reasonable. Earthquake records obtained in the basement of buildings will include implicitly all three effects. The first one can be eliminated if simultaneous records are obtained on the free surface at a sufficient distance from the building to have properly free field conditions. Computing then the Fourier transforms of the motions at the base of the building (say $\ddot{u}_D$) and on the free surface ($\ddot{u}_A$) and dividing the first by the second at each frequency one would obtain the transfer function from A to D. This will be a complex function of frequency with both real and imaginary parts, or, more conveniently, an amplitude and a phase (both functions of frequency). Separation of the two interaction effects requires, on the other hand, knowledge of the properties of the structure. If K, C, M are the stiffness, damping, and mass matrices of the structure, including the foundation, one can define at each frequency Ω a dynamic stiffness matrix

$$S = K + i\Omega C - \Omega^2 M \quad (1)$$

Denoting by a subscript 2 the degrees of freedom corresponding to the point of contact between the foundation and the soil and by a subscript 1 all other degrees of freedom of the structure and letting S_f be the dynamic stiffness matrix of the soil-foundation interface (relating harmonic forces and displacements of the soil along this interface) the equations of motion of the soil-structure system can be written in the frequency domain as

$$\begin{bmatrix} S_{11} & S_{12} \\ S_{21} & S_{22}+S_f \end{bmatrix} \begin{Bmatrix} U_1 \\ U_2 \end{Bmatrix} = \begin{Bmatrix} o \\ S_f U_3 \end{Bmatrix} \quad (2)$$

where U_2 are the displacements at the interface accounting for the presence of the structure and U_3 the displacements of a massless foundation (without structure). For a rigid foundation matrices S_{22} and S_f will be in general 6x6 relating forces (and moments) to displacements (and rotations) at the centroid of the base contact area. The displacements U_2 become the displacements of point D in figure 1, while U_3 are the displacements U_C. Then

$$U_C = S_f^{-1} [S_{22} + S_f - S_{21} S_{11}^{-1} S_{12}] U_D \quad (3)$$

or

$$U_C = U_D + S_f^{-1} [S_{22} - S_{21} S_{11}^{-1} S_{12}] U_D \quad (4)$$

The term $[S_{22} - S_{21} S_{11}^{-1} S_{12}] U_D$ are clearly the forces at the base of the structure (point D) caused by the structural vibrations due to the base displacements U_D. When multiplied by the inverse of the soil stiffness matrix they become the additional soil displacements due to these forces.

If motions are recorded at the basement of a building equations 3 or 4 allow computation of the motions that would have occurred at the base of a massless foundation without the structure. Knowing the transfer functions from point A (free surface of the soil) to point D one could then obtain the transfer functions from A to C, isolating the kinematic interaction effects. It should be noticed that the transfer functions for displacements are also the transfer functions for accelerations.

To illustrate the differences between the transfer functions from A to D (including the combined effects of kinematic and inertial interaction) and from A to C (accounting only for kinematic interaction) figure 2 shows the amplitudes of these two transfer functions for a short building with a fundamental frequency (on a rigid base) of 3 Hz, on a medium to soft soil with a ratio of the embedment depth E to the radius of the foundation R of only 0.125. The seismic excitation is assumed to consist of vertically propagating shear waves (the most common assumption in seismic analyses). It can be seen that the amplitude of the horizontal motion at point G (kinematic effects) varies from 1 to 0.75 times the corresponding amplitude at the free surface (point A). Inertial interaction effects cause a significant additional reduction with pronounced valleys at the natural frequencies of the structure. There is on the other hand a small increase of the amplitude of motion at the fundamental frequency of the combined soil structure system. Clearly in this case attempting to backfigure kinematic interaction effects from motions recorded at the base of the building, without eliminating the inertial interaction effects, would result in an unconservative overprediction of embedment effects.

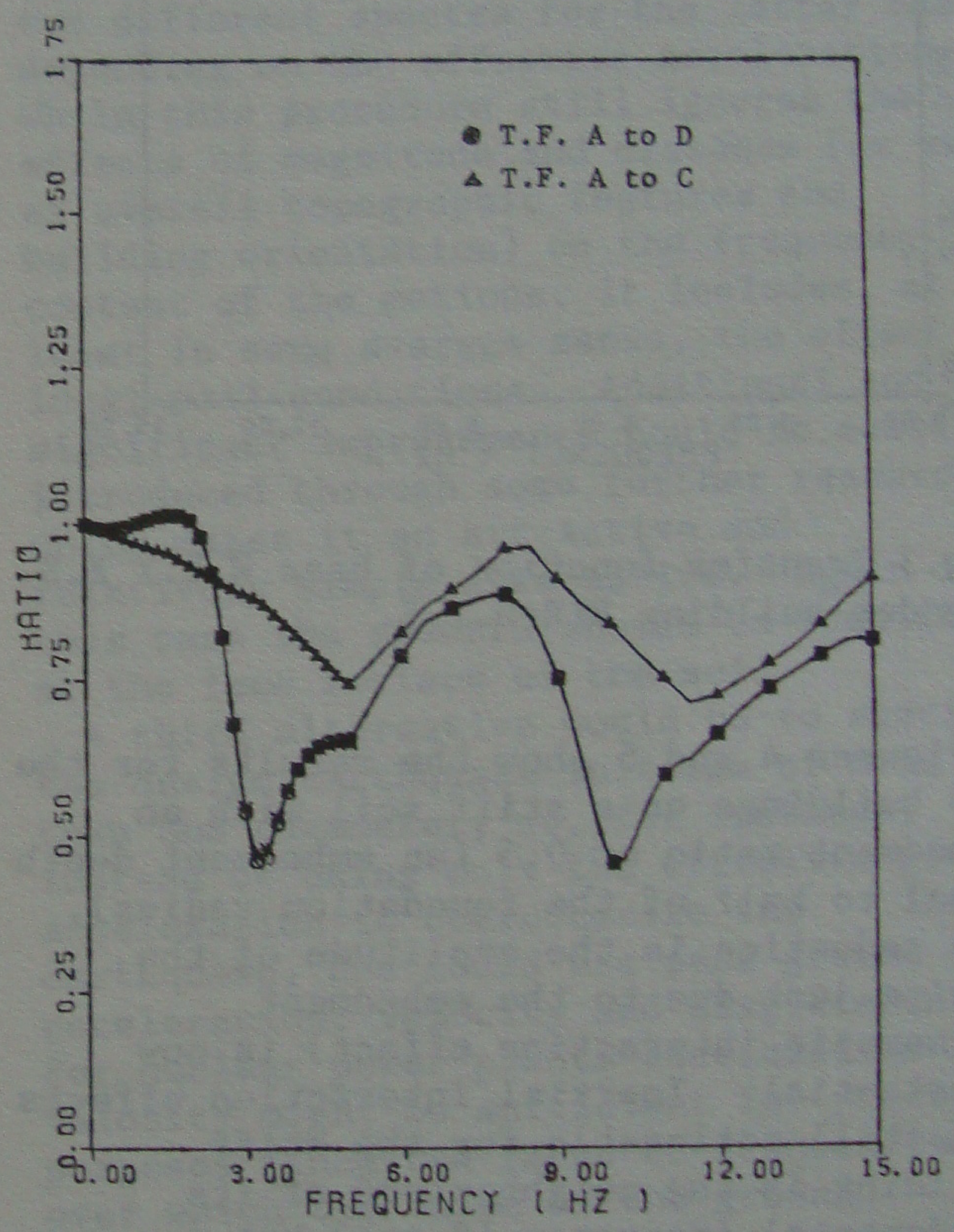

Fig.2 Transfer function of base W.R.T F.S. short building E/R=0.125.

Figure 3 shows the corresponding results for a very flexible and slender building with a fundamental frequency (on a rigid base) of 0.33 Hz, the same medium to soft soil and an identical embedment ratio. The transfer function from A to C is the same as in the previous case since it is independent of the structure. The transfer function from A to D shows again some additional reduction in the amplitude of motion due to inertial interaction effects but the reduction is now much smaller (of the order of 10%). The transfer function exhibits also a large number of oscillations associated with the natural frequencies of the structure.

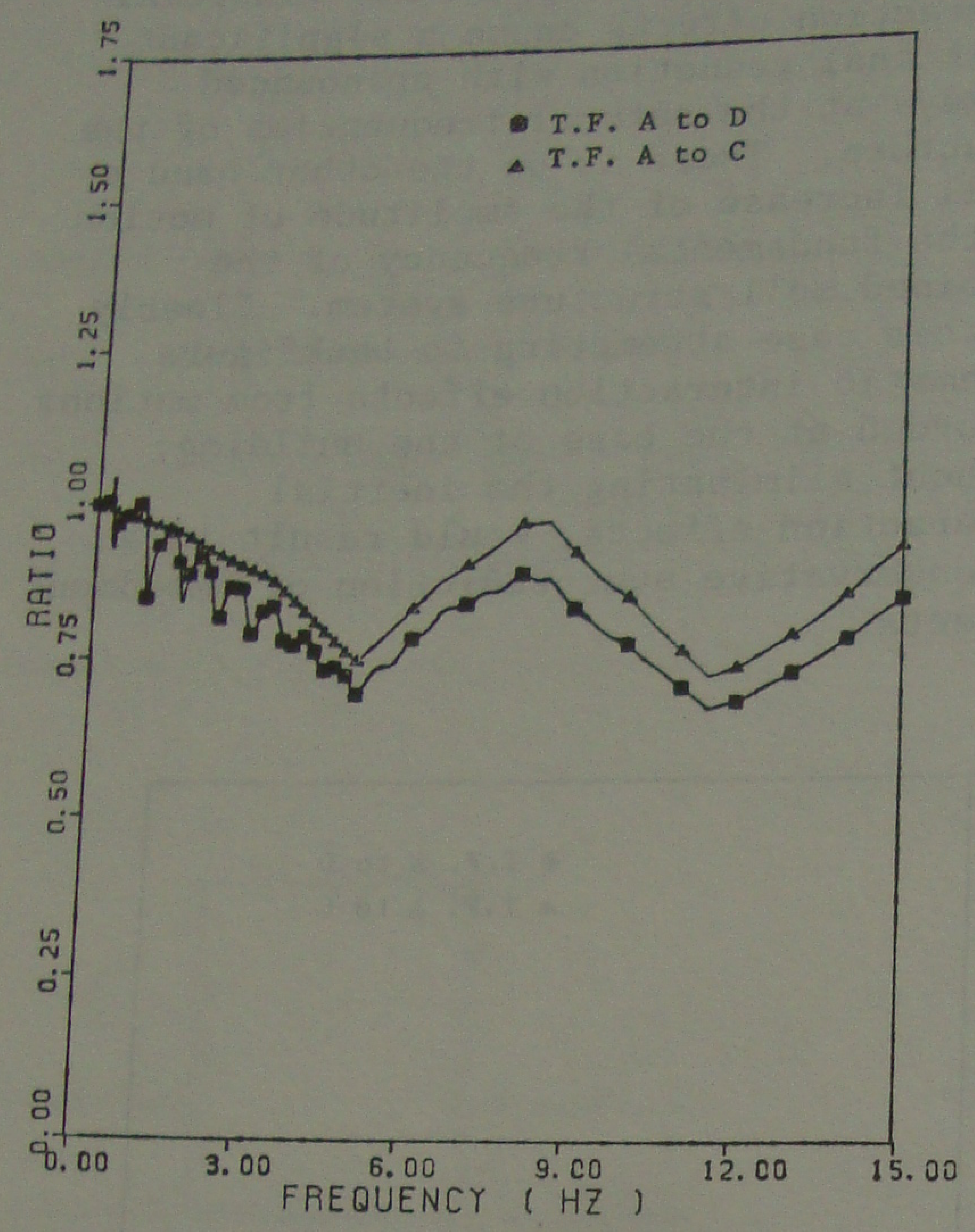

Fig.3 Transfer function of base W.R.T F.S. slender building E/R=0.125.

Figures 4 and 5 show the results for the two buildings on a stiff soil with an embedment ratio of 0.5 (an embedment depth equal to half of the foundation radius). The reduction is the amplitude of the motion just due to the embedment (kinematic interaction effect) is now substantial. Inertial interaction effects are still noticeable for the stiff building in the neighborhood of its fundamental frequency (3 cps) but are very small outside of this range. For the flexible building inertial effects are negligible and assuming that the motions recorded at the basement of the building are the ones that a massless foundation would experience would be reasonable.

Clearly if one knew the motions at point D one could perform the seismic analysis of the structure using classical programs of dynamic analysis, which ignore soil structure interaction effects, without any need to compute the motions at C. In

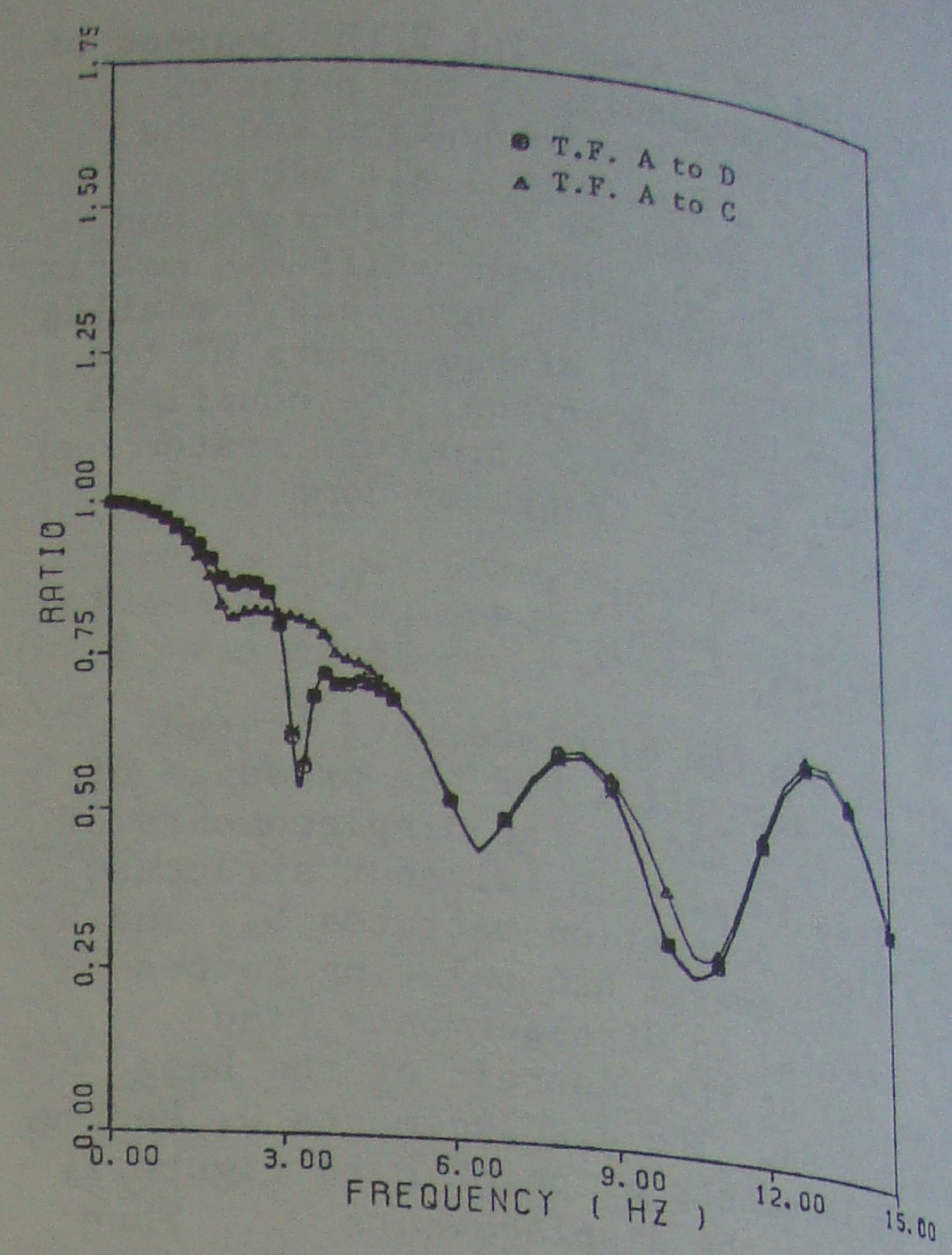

Fig.4 Transfer function of Base W.R.T F.S. short building E/R=0.500.

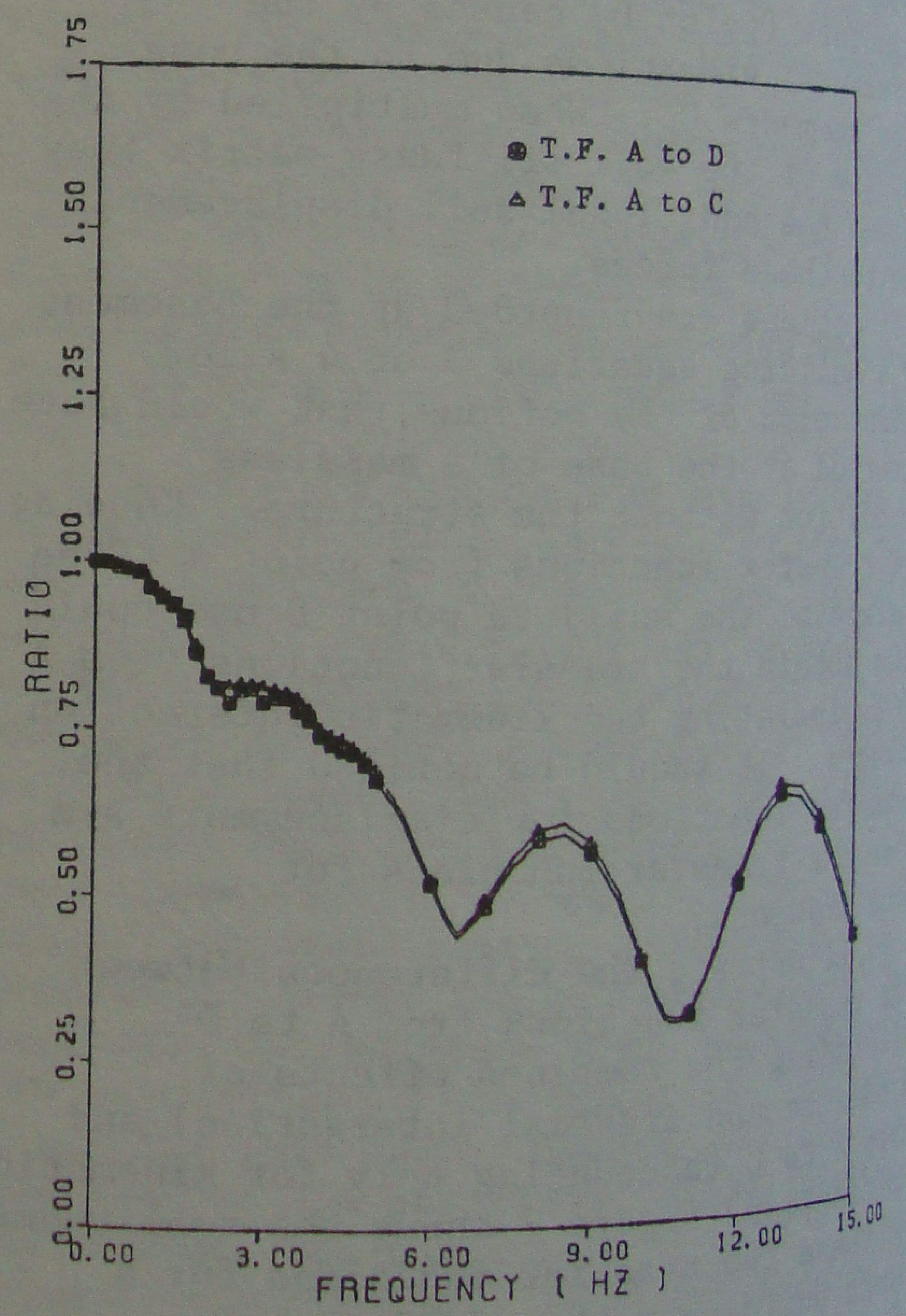

Fig.5 Transfer function of base W.R.T F.S. slender building E/R=0.500.

practice, however, one would know the characteristics of the design earthquake in the free field. Using the three step, or substructure, approach for the seismic analysis of the structure, one would proceed to compute the dynamic stiffness matrix of the foundation S_f and the motions of a massless foundation U_C. Finally the dynamic analysis of the structure would be performed using equation 2 to incorporate inertial interaction effects.

This paper is concerned primarily with the effect of embedment on the motions of a massless foundation (kinematic interaction) and its relationship to the location at which the control design motion is specified.

2 SPECIFICATION OF THE SEISMIC INPUT

It is common practice today to specify the seismic input in terms of a peak ground acceleration (or an effective ground acceleration) and a standard (generic) family of response spectra for various values of structural damping. Artificial earthquakes are then generated: two with the same intensity for two perpendicular horizontal directions and a third one, scaled by a factor of 2/3 for the vertical component. These synthetic records can then be used for time history analyses. This procedure ignores clearly the effects of the earthquake mechanism, magnitude and distance, the general topographic and geologic features between the fault and the site, local soil conditions and orientation of the building with respect to the fault on the frequency content of the motions. Because the R.G. 1.60 rules to construct design spectra are based on a statistical analysis of a large number of actual earthquake records, corresponding to different conditions (magnitude, distance, soil profiles) they are supposed to provide, within a certain probability level, a conservative upper bound to the motions that can be expected at any site. If this conservatism applied not only to the design spectra but also to the results of the soil structure interaction analyses, while the SSI effects might be substantially different for actual or site specific records, the use of the synthetic accelerograms based on the R.G. 1.60 spectra would be a reasonable choice when more information is not available, particularly for soil deposits that can be classified within the all encompassing and conveniently vague term of firm ground. The design spectrum should then apply at the free surface of the soil deposit. Some recent studies by Luco et al (1985) have shown that results of SSI analyses with some actual records may be larger than those obtained with artificial earthquakes based on R.G. 1.60 generic spectra with the same effective acceleration. The reasons for these larger values need further clarification. Apart from this question the main problem with this procedure is that the degree of conservatism involved is hard to quantify: it may be very large in some occasions and nonexistent in others leading to a lack of uniformity in safety requirements.

A variation from the previous approach that represents an improvement is to use more than one standard spectrum for a given level of acceleration. Following, for instance, the recommendations of the ATC-3 one could specify a spectrum (or a family of spectra for various values of damping) for rock, another one for firm ground and a third one for very deep soil deposits. The ATC-3 recommends in fact two different spectra for the latter case, depending on the effective acceleration. While this procedure still ignores the effects of magnitude and distance (as well as overall topographic features and building orientation) on the frequency content of the motions, it includes, at least in some average sense, the effect of local soil conditions. Additional and significant improvements could be easily introduced through some further research, which makes it an attractive and relatively simple alternative. Again in this case the spectra should be specified at the free surface of the soil.

A third alternative would be to specify the design earthquake in terms of more than one parameter; so, for instance, instead of using only the effective ground acceleration to characterize the earthquake, one could use peak ground acceleration, velocity and displacement. For nuclear power plants acceleration and velocity might be sufficient for practical purposes since the range of frequencies over which the value of the ground displacement is important is rarely of interest. Smooth design response spectra based on these three quantities could then be derived, for different values of damping, using rules like those suggested by Newmark (1967), García (1970), Newmark, Hall and Morhaz (1973) or Riddell and Newmark (1979). This approach has the potential to include many of the effects neglected in the previous two if one could find appropriate functional relationships between peak ground acceleration, velocity and displacement and earthquake mechanism, magnitude, distance, soil conditions etc. Research in this area using both empirical data and analytical earthquake models could greatly improve the state of the art and make this approach very attractive. The corresponding earthquake parameters and the resulting spectra should apply, once more, at the free surface of the soil.

An even simpler approach would be to use real earthquake records corresponding to magnitudes, distances and general and local soil conditions similar to those of the situation at hand. These records could then be used directly, performing the analyses for various motions instead of a single one, and interpreting statistically the result, or could serve as a basis to construct appropriate, smooth, design spectra. In both cases the motions would apply again at the free surface of the soil. The main problem with this approach is the availability of a sufficient number of actual records to provide an adequate data base. Synthetic accelerograms based on physical models of the rupture along the fault can be used to complement recorded data. On the other hand scaling site specific records to a desired acceleration does not provide a very accurate or reliable means to increase the existing data base.

All four of the above approaches would specify the design earthquake in terms of response spectra or time histories of acceleration at the free surface of the soil. None of them requires an explicit definition of the type of seismic waves that generate the motions, although this information is always necessary for kinematic interaction analyses (to determine the motions around the foundation). A slightly different alternative is to specify the design spectrum at the free surface of a hypothetical rock outcropping. For a soil profile where bedrock can be clearly identified at a finite depth, wave propagation analyses can then be conducted to determine compatible motions at any point in the free field. This approach allows therefore construction of site specific spectra for each individual case. It requires, however, an explicit indication of the types of waves that constitute the earthquake. At present this method is used in practice for shallow or only moderately deep soil deposits assuming vertically propagating shear waves for the horizontal motions and compressional waves for the vertical component. When the depth to bedrock is not clearly defined, or is very large, the use of this procedure is less attractive and the definition of an alternative spectrum for deep soil deposits, as in the second approach discussed, would be more appropriate.

A final alternative would be to specify the design motions in the free field in terms of wave amplitudes and angle of incidence as a function of frequency. While this would be the most desirable approach for soil structure interaction analyses a complete definition of the earthquake this way may be extremely difficult and cumbersome. It would be necessary to define where these waves and amplitudes apply. The most logical approach would be in rock but this would require again, as in the previous approach the existence of bedrock at a finite, and not too large, depth. Moreover if there is internal dissipation of energy in the soil, as would be normally the case, the horizontal location would also have to be specified since the amplitudes would decay with horizontal distance (this may be an important consideration when analyzing simultaneously various buildings). In spite of these difficulties some simplified form of this approach is possible and is needed in all cases for kinematic interaction studies, to investigate the effect of adjoining structures and for the seismic analysis of bodies with large dimensions (pipes and conduits, dams, long bridges, very large mats supporting more than one building, etc.).

3 LOCATION OF THE CONTROL MOTION

Six different alternatives have been suggested in the past for the location where the control motion is to be specified. They are:

- Free surface of the soil deposit at the site
- Free surface (outcropping) of rock
- Foundation level of massless foundation, without structure
- Foundation level of structure
- Bedrock (interface between soil and rock at the site)
- Foundation level in the free field.

Of these six possibilities the first two are the most logical ones. As discussed in the previous section the design motion should be specified at the free surface of the soil when the input is defined by a generic (R.G. 1.60) type of response spectra, by a set of standard site specific spectra, by the values of the peak ground acceleration, velocity and displacement at the site (including effects of local soil conditions) or by a collection of real earthquake records corresponding to similar conditions (the first four approaches). The second alternative corresponds to approaches 5 and 6 in the specification of the design motion. It is particularly appropriate for soil deposits of moderate thickness where the location of the bedrock can be clearly identified and it allows to conduct wave propagation studies to determine compatible site specific free field motions.

The third alternative would ignore entirely kinematic interaction effects due to embedment and shape of the foundation. For surface foundations and vertically propagating waves it is the same as the first one. For embedded foundations it will result normally in larger translational motions than could be realistically expected and in the absence of rotational components that would actually occur. It implies basically that the specified design motion is the motion of point C in figure 1, U_C. This alternative not only ignores theoretical knowledge and physical evidence but it provides a degree of safety or conservatism which is hard to quantify; it can be very large (and excessive) for some situations and nonexistent for others. In addition it gives rise to serious inconsistencies when dealing with several buildings founded at different depths (or with different types of foundations) and attempting to study the effects of adjoining buildings or the behavior of connecting elements (pipes, conduits, etc.). Clearly an analysis that would model together the different structures and the surrounding soil would be meaningless with the motions specified independently for each foundation. Analyses with travelling waves (instead of simply vertically propagating waves) would be equally devoid of any reasonable meaning.

The fourth alternative would ignore all soil structure interaction effects (both kinematic and inertial interaction) and would represent a return to the approach of the past where structures were assumed to be on a rigid base. In this case the specified motion would be that of point D in figure 1, U_D. Neglecting soil structure interaction effects may be reasonable for structures founded on rock subject to horizontal excitation (for vertical motions inertial interaction effects may still be important due to the higher frequencies of interest and the large values of the radiation damping). For other conditions and for very stiff and massive structures such as nuclear power plants ignoring SSI effects is equivalent to negating the research accomplishments of the last twenty years. Moreover, while the approach would be generally conservative when using generic R.G. 1.60 spectra (and the conservatism may be extreme in some cases), it could be unconservative when using site specific spectra or real earthquake records.

The fifth and sixth alternatives correspond to specification of the seismic input at points within the soil profile in the free field. Since the motions at these points are a function of the soil properties these approaches can only be applied when the input is specified in terms of the amplitudes and angles of the seismic waves (the last procedure discussed in the previous section), and wave propagation analyses are conducted for the site. Specification of the motion at bedrock (fifth alternative) would be a reasonable alternative if the input was specified in terms of a rock spectrum and the modulus of the soil was substantially smaller than that of the rock. It would then be nearly equivalent to specifying the motion at rock outcropping (the second alternative). For all other cases, when the input is defined in terms of design spectra (generic or site specific) or in terms of actual earthquake records these two approaches lack any logic and should be avoided. They would imply highly unreasonable motions at the free surface of the soil deposit and at other levels than the one where the motion is specified and would make again meaningless the simultaneous study of various buildings. The fifth alternative would correspond in fact to direct specification of the motion at point B of figure 1. Unfortunately this is the option that was recommended and even required at times in the recent past. It should be noticed that these last alternatives do not eliminate the kinematic interaction analyses (as would alternatives three and four). They just make the results of these analyses rather meaningless from a physical point of view. Yet the requirement that the control motion be specified at the foundation level in the free field was based supposedly on the desire to eliminate wave propagation studies and to avoid having a motion at the base of the structure with deep valleys in the response spectrum at frequencies which may coincide with those of the structure or the equipment.

Clearly if a motion with a smooth spectrum is specified at the free surface of a soil deposit the spectra of the corresponding motion at a depth E in the free field (point B in figure 1) will exhibit some pronounced valleys, particularly at the fundamental frequency of the soil layer between levels B and A, as illustrated in figure 6. On the other hand the spectra of the compatible translational motion at the base of a massless foundation (point C), resulting from an appropriate kinematic interaction analysis, will have a smaller valley (at a frequency different from the fundamental frequency of the embedment layer) and will be much smoother overall. There will be in addition rotational components of motion at point C. If the same control motion had been specified at point B the resulting motions at the free surface (point A) and at the base of the massless foundation (point C) would have a very large and entirely unrealistic peak in their response spectra at the frequency of the soil layer between A and B.

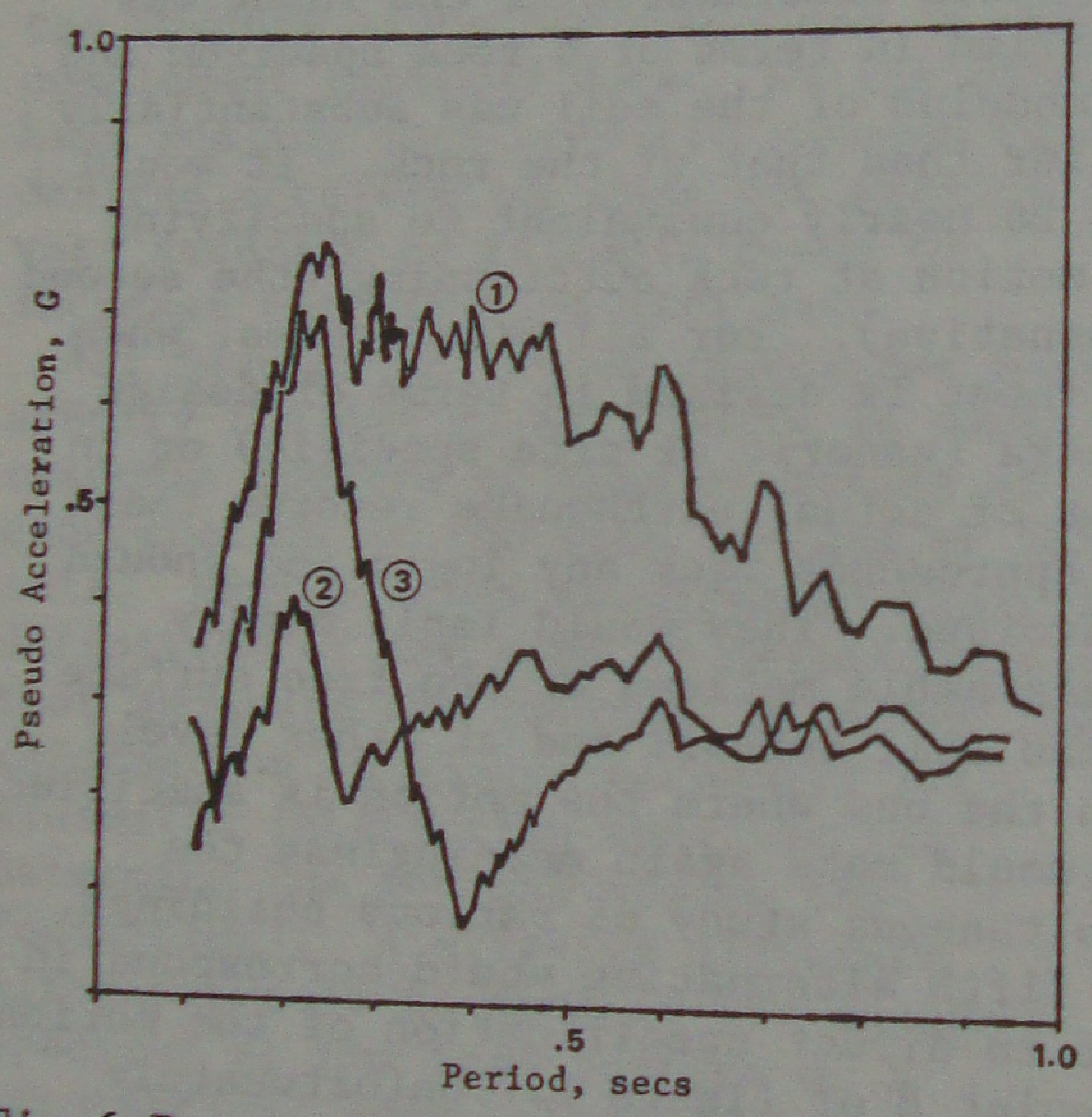

Fig.6 Response spectra of motions at
1 free surface of soil
2 foundation (with kinematic interaction)
3 foundation level free field.

The apparent confusion between the motions of an embedded foundation and those that would occur at the foundation level in the free field may have been aggravated by the large number of studies conducted to justify the reduction in acceleration levels for embedded foundations using only one dimensional wave propagation analyses in the free field (typically deconvolution analyses assuming vertically propagating waves) and comparing theoretical predictions with motions recorded at different depths, instead of using proper kinematic interaction procedures. While these studies are valuable to justify the techniques used for wave propagation analyses in the free field and even the use of the simple one dimensional theory for horizontally stratified soil deposits, they cannot be directly applied to the study of foundation motions. It should be noticed that not only do these include rotational components which are not present in the free field but the transfer function from A to B (free field) is only a function of the soil properties, the types of waves and the depth E, while the transfer function from A to C depends also on the shape of the foundation (in particular the ratio of the embedment depth E to the radius R or an equivalent base dimension). As will be shown in the following as the size of the foundation increases, for a fixed embedment depth, the reduction in the translational motion from A to C decreases.

In spite of the resistance it has encountered among regulatory agencies the most logical approach is to specify the control motion at the free surface of the soil deposit or at a hypothetical outcropping of rock and to conduct appropriate kinematic interaction analyses to compute consistent foundation motions. Although an assumption has to be made as to the type of waves, considering the simplest case of vertically propagating waves should provide in general reasonable results (other types of waves may be considered in special circumstances). A legitimate source of concern is the fact that the details of the motions of the foundation and their frequency content (in particular the valleys in the transfer functions and the response spectra) are a function of the wave content of the earthquake and the soil properties. Clearly if the analyses are performed for a single earthquake record, whether real or artificial, a unique type of waves and a single set of soil properties the results could be unconservative. On the other hand when analyses are performed for three sets of soil properties (as is often required) the location of the valleys will be shifted and the envelope of the final results should eliminate potential unconservatisms. Various approaches can be followed to account for the uncertainties in the types of waves, earthquake characteristics and soil properties:

- to limit the amount of reduction in the translational motions of the foundation with respect to those at the free surface at each frequency. While this is not an optimum solution because it does not address the rotational (rocking or torsional) motions, it is reasonable in the absence of more detailed information.

- to conduct the analyses not only for three different sets of soil properties but also for more than one train of waves (if a small number of logical trains can be defined) and for more than one earthquake (particularly when dealing with actual records) and to envelop the results. If the results to be enveloped are the motions at the foundation the approach does not offer significant advantages over the previous one: it would be possible to envelop the response spectra of the translational motions and then to generate a synthetic earthquake to match the smoothed spectra but it is not easy to do the same with the rotational components of motion and still get reasonable results since the phases between the different components, which are important, would be lost. The most sensible approach would be to envelop, or interpret statistically, the results of the final interaction analyses. The main objection to this procedure is the large number of different analyses that would have to be conducted and the associated cost. While this cost may not be as large as claimed if one uses a modal synthesis of the structure it is clear that some compromise must be reached as to the total number of combinations of soil properties and types of waves that should be studied. The use of a probabilistic formulation, both in the definition of the earthquake and in the dynamic analyses accounting for soil structure interaction effects would also help to reduce the number of computations providing an indication of the variability inherent in the results.

4 DETERMINATION OF FOUNDATION MOTIONS

The input for an inertial soil structure interaction analysis (the last step of the three step or substructure approach) must be the motions that would occur at the interface between the foundation and the soil before the structure is built but accounting for the geometry of the excavation (and the foundation stiffness). For a flexible foundation, as explained earlier, there are three translational components of motion at each interface point. For a rigid foundation, when these interface motions are condensed by imposing the constraints of a rigid body there will be in general three translational and three rotational components of motion. An explicit determination of the foundation motions is not necessary in a direct soil structure interaction analyses (when the structure and the soil are modelled together) or even in a substructure analysis if the right hand side of the equations of motion (equation 2) is expressed in terms of the motions at the foundation-soil interface in the free field, without any excavation, and a stiffness matrix for the interface points also in the free field. In both cases, however, kinematic interaction effects would automatically be included.

The foundation motions before the structure is built can be determined with the same techniques used to compute dynamic stiffnesses of foundations or to perform direct soil structure interaction analyses. There is a persistent misconception that discrete formulations (such as finite elements) can only be used for shallow soil profiles underlain by rigid rock, while the impedance approach is only applicable when dealing with an elastic half space. This mistake is apparently caused by the identification of each of these procedures with past studies or applications in their simplest form rather than a clear understanding of the fundamentals and general potential of these methods. Whether the discretization extends over a finite soil domain with consistent lateral boundaries (as in finite element models) or it is applied only to the interface between the foundation and the soil using appropriate Green's functions and an integral equation formulation (with weighted residuals, direct or indirect boundary element method) it is possible to obtain solutions to dynamic soil structure interaction problems with any desired degree of accuracy. The only differences between these two approaches, when properly implemented, are those of convenience, availability of computer codes and economics. A solution based on the use of the continuous Green's functions is particularly appropriate when the soil can be considered as an elastic half space or can be reproduced by a small number of horizontal layers. Finite element type formulations or discrete Green's functions tend to be advantageous when a large number of layers is needed to reproduce properly the variation of soil properties with depth. Both approaches become more complicated and expensive when dealing with nonlinear problems (nonlinear soil behavior or nonlinear effects at the foundation-soil interface), soil deposits where the properties vary in the horizontal direction as well as with depth, or when the stratum is not horizontally layered (sloping layers or bedrock, two or three dimensional geometries), although a finite element model is less affected by some of these variations.

Whatever the method used the determination of the foundation motions without structure requires the specification of the types of waves that give rise to the motions in order to compute the displacement U_o at the foundation-soil interface in the free field (and possibly also the tractions T_o on the soil at this interface). When the earthquake is not defined directly in terms of its wave content it will be necessary to make explicit or implicit assumptions in this respect. The most common assumption is that the seismic waves are propagating vertically although special analyses are also conducted at times for horizontally travelling waves or surface waves. The consideration of any wave train (or a combination of wave fronts) offers no special difficulty.

For a finite element type formulation let U_o represent the nodal displacements of the soil in the free field along the surface that will eventually constitute the interface with the foundation (see figure 7) and F_o the corresponding nodal forces (resultants of the tractions) acting on the soil. Let S_o denote the dynamic stiffness matrix of the soil-foundation interface in the free field (relation between forces and displacements at the nodes along the interface, S_f the dynamic stiffness matrix for the same points but accounting for the excavation and S_e the dynamic stiffness matrix of the soil mass to be excavated. Clearly

$$S_o = S_f + S_e \tag{5}$$

Considering now the soil with the excavation, but without any foundation, the displacements along the surface of the excavation can be expressed as a combination of the free field motions and the displacements due to forces acting on the excavation equal and opposite to F_o so that the net forces are zero. Then

$$S_f(U - U_o) = -F_o \tag{6}$$

or

$$U = U_o - S_f^{-1}F_o \tag{7}$$

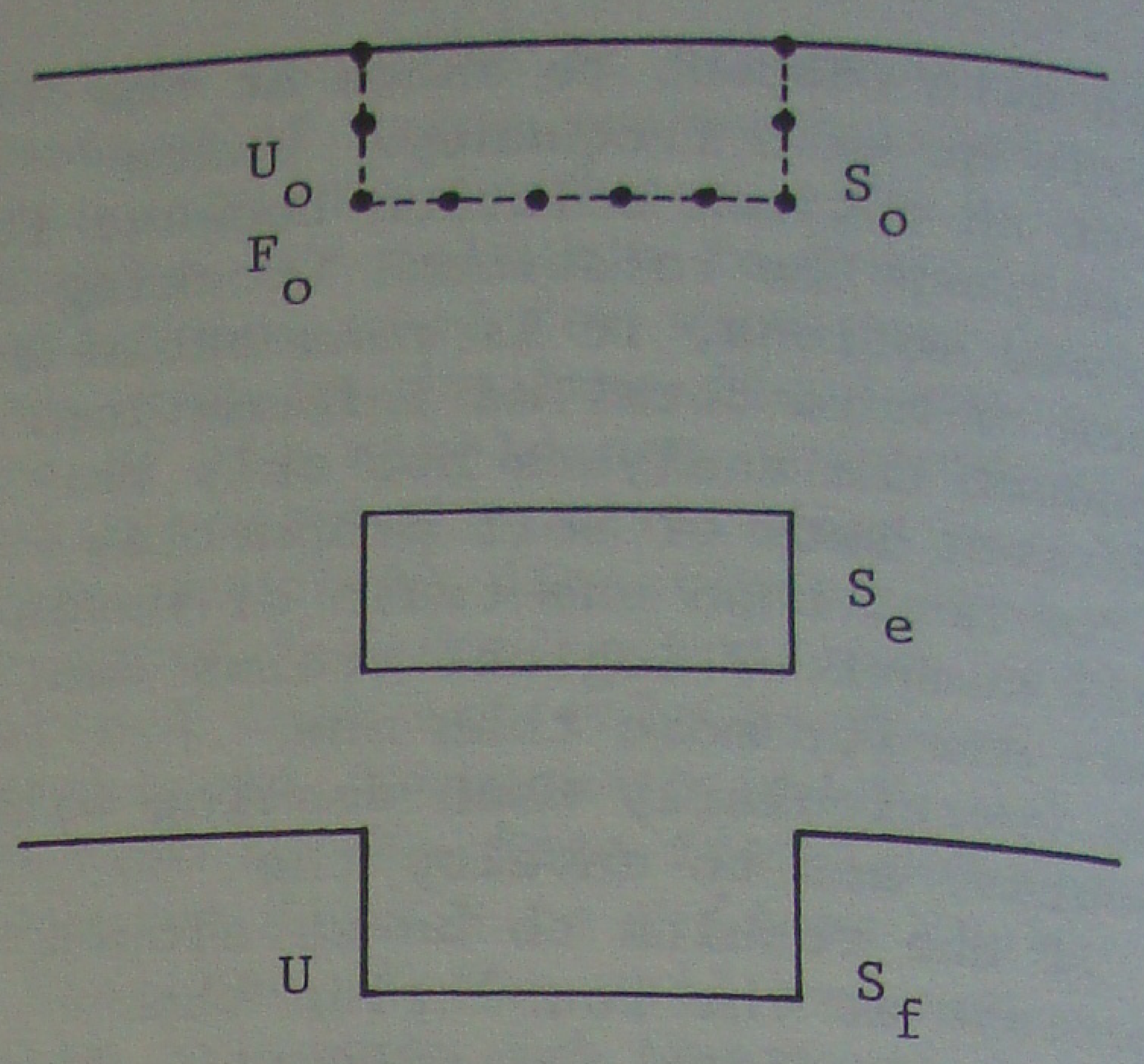

Fig.7 Definition of displacements and stiffnesses

Taking on the other hand the soil mass to be excavated by itself one would have

$$S_eU_o = -F_o \tag{8}$$

so that expression (6) can be written alternatively as

$$S_fU = S_fU_o + S_eU_o = S_oU_o \tag{9}$$

Notice that for a flexible foundation if the mass and stiffness characteristics of the foundation are included in the model of the structure the right hand side of the equations of motion (2) is S_fU. One can therefore replace it by S_oU_o without the need to compute explicitly U (one must on the other hand compute in this case the matrix S_o).

For a rigid but massless foundation the displacements U along the faces of the excavation can be related to three displacements and three rotations along the centroid of the base contact area (point C) in the form

$$U = LU_C \tag{10}$$

The nodal forces F on the interface have resultant forces and moments at C

$$P_C = L^TF \tag{11}$$

which must be zero.

Then

$$(L^TS_fL)U_o = L^TS_fU_o - L^TF_o \tag{12}$$

or

$$(L^TS_fL)U_C = L^TS_oU_o \tag{13}$$

where L^TS_fL is the dynamic stiffness matrix of the rigid foundation to be used also for the inertial interaction analysis.

Using instead a boundary element formulation let U_o denote again the displacements in the free field at the eventual soil-foundation interface due to the seismic waves and T_o the corresponding tractions acting on the soil (the values of these tractions at the nodal points) and U, T the final nodal values of the displacements and tractions. Assuming a family of interpolation functions so that the displacements and tractions at any point u, t are related to their nodal values by

$$u = NU \tag{14}$$

$$t = NT$$

the equivalent nodal forces F, F_o of the previous formulation would be obtained integrating the tractions multiplied by the interpolation functions

$$F = \int N^T t = (\int N^T N)T = AT \tag{15a}$$

$$F_o = \int N^T t_o \tag{15b}$$

In the direct boundary element method the nodal displacements U and the nodal values of the tractions T acting on the soil along the faces of the excavation are related by

$$HU = GT \tag{16}$$

where the terms of the matrix H are obtained integrating over each boundary element the tractions acting on the soil interface in the free field due to concentrated unit forces at each node multiplied by the interpolation functions and the terms of the matrix G result from integration of the corresponding displacements multiplied by the interpolation functions. Then from expression (16)

$$T = G^{-1}HU \tag{17}$$

and

$$F = AG^{-1}HU = S_f U \tag{18}$$

so that the product $AG^{-1}H$ would be equivalent to the dynamic stiffness matrix S_f of the excavation as defined before. When considering the effect of the seismic waves in the free field equation (16) becomes

$$H(U - U_o) = G(T - T_o) \tag{19}$$

or

$$U = U_o - H^{-1}GT_o \tag{20}$$

since the total tractions T along the excavation must be zero. This is the equivalent of the previous equation (7). By considering again the soil mass to be excavated by itself it can be easily shown that

$$HU_o - GT_o = U_o \tag{21}$$

so that equation (20) can also be written as

$$HU = U_o \tag{22}$$

or

$$U = H^{-1}U_o \tag{23}$$

For a rigid foundation writing equation (11) in the form

$$P_C = L^T AT \tag{24}$$

and making use of equation (10)

$$(L^T AG^{-1}HL)U_C = L^T A(G^{-1}HU_o - T_o) \tag{25}$$

or

$$(L^T AG^{-1}HL)U_C = L^T AG^{-1}U_o \tag{26}$$

where $L^T AG^{-1}HL$ is the equivalent of the dynamic stiffness matrix of the rigid foundation.

Alternatively using a formulation as suggested by Wolf (1985) one can express the displacements and tractions along the surface of the excavation as

$$u = u_o + G_u Q \tag{27}$$

$$t = t_o + G_t Q \tag{28}$$

where Q are concentrated or distributed forces applied at an infinitesimal distance from the interface on the soil to be excavated and G_u G_t are matrices containing the Green's functions for the displacements and tractions at any point on the interface in the free field due to unit values of the loads (or load densities) Q.

Assuming again a matrix of interpolation functions N so that the displacements u and tractions t can be expressed in terms of their nodal values U, T as

$$u = NU \tag{29}$$

$$t = NT \tag{30}$$

the equivalent nodal forces can again be defined by equation (15).

Weighted residual techniques can then be applied to equations (27) and (28) in a variety of ways. Using for instance the interpolation functions N as weighting functions for equation (28) and enforcing the condition of zero tractions at the surface of the excavation

$$(\int N^T G_t)Q = -\int N^T t_o = -F_o \qquad (31)$$

which allows computation of the unknown loads Q. The matrix $\int N^T G_t$ is the transpose of the matrix H of equations (16) to (20).

Applying then collocation to equation (27)

$$U = U_o - G_u H^{T-1} F_o \qquad (32)$$

where the terms of the matrix G_u are evaluated at the nodal points.

Wolf (1985) suggests instead the use of a weighting family of functions G_t for equation (27) leading to

$$(\int G_t^T N)U = \int G_t^T u_o + (\int G_t^T G_u)Q \qquad (33)$$

and making use of equation (32)

$$U = (\int G_t^T N)^{-1} [\int G_t^T u_o - (\int G_t^T G_u)(\int N^T G_t)^{-1} F_o] \qquad (34)$$

For the rigid foundation the corresponding results would be

$$[L^T(\int N^T G_t)G_u^{-1} L]U_C = L^T(\int N^T G_t)G_u^{-1} U_o + L^T \int N^T t_o \qquad (35)$$

using collocation, and

$$[L^T(\int N^T G_t)(\int G_t^T G_u)^{-1}(\int G_t^T N)L]U_C = L^T(\int N^T G_t)(\int G_t^T G_u)(\int G_t^T G_u)(\int G_t^T u_o) - L^T \int N^T t_o \qquad (36)$$

with the other weighting functions.

A simpler approach, which is not exact but provides an excellent approximation, has been proposed by Iguchi (1982) for rigid foundations. Calling now S_f the dynamic stiffness matrix of the rigid foundation (defined earlier as $L^T S_f L$) and L the matrix relating the displacements at one point of the surface of the excavation to those of point C

$$u = LU_C \qquad (37)$$

where L is now 3x6 and a function of the coordinates of the selected point. The forces and moments around C caused by the tractions t along the excavation would be

$$P_C = \int L^T t \qquad (38)$$

Calling again u_o t_o the displacements and tractions acting on the soil interface surface in the free field (without excavation) due to the seismic waves and defining a matrix

$$B = \int L^T L \qquad (39)$$

the displacements U_C are given in the approximate solution by

$$U_C = B^{-1} \int L^T u_o - S_f^{-1} \int L^T t_o \qquad (40)$$

In all the above expressions the integrations are performed over the surface of the excavation (or the soil-foundation interface).

The first partial studies on kinematic interaction effects caused by travelling waves were presented by Newmark (1969), who pointed out the existence of torsional components of motion for rigid surface foundations subjected to horizontally propagating shear waves, and Yamahara (1970), who pointed out that for this same case there will be a filtering effect of the translational motion. The motions of embedded foundations caused by vertically propagating waves (the main topic of interest in this paper) were studied by Elsabee and Morray (1977) who suggested approximate expressions for the transfer functions for the horizontal translation and rotation of a rigid circular foundation for a specified motion at the free surface of the soil deposit (figure 8). As can be seen the translational motion decreases in amplitude with increasing frequency up to a frequency which is about 0.7 times the natural frequency of the embedment layer and fluctuates for larger frequencies about a constant average value which is about 45% of the amplitude at the free surface. For relatively flexible structures on stiff soils or with small embedment the reduction in the motion at the main frequency of interest (fundamental frequency of the soil-structure system) will be very small and often negligible. For a stiff structure with significant embedment in relation to the base dimension and a soft soil the frequency of interest can be in the range of maximum reduction and the effect would be significant. It should be noticed, however, that the reduction in the translational motion is accompanied by the creation of a rocking component. The amplitude of the base rotation increases with frequency up to about the natural frequency of the embedment layer, then remains nearly constant with some fluctuations (over the range of frequencies studied). The average amplitude of the vertical motion at the edge of the foundation due to this rotation in the range where it is nearly constant is about 25% of the amplitude of the horizontal motion at the free surface.

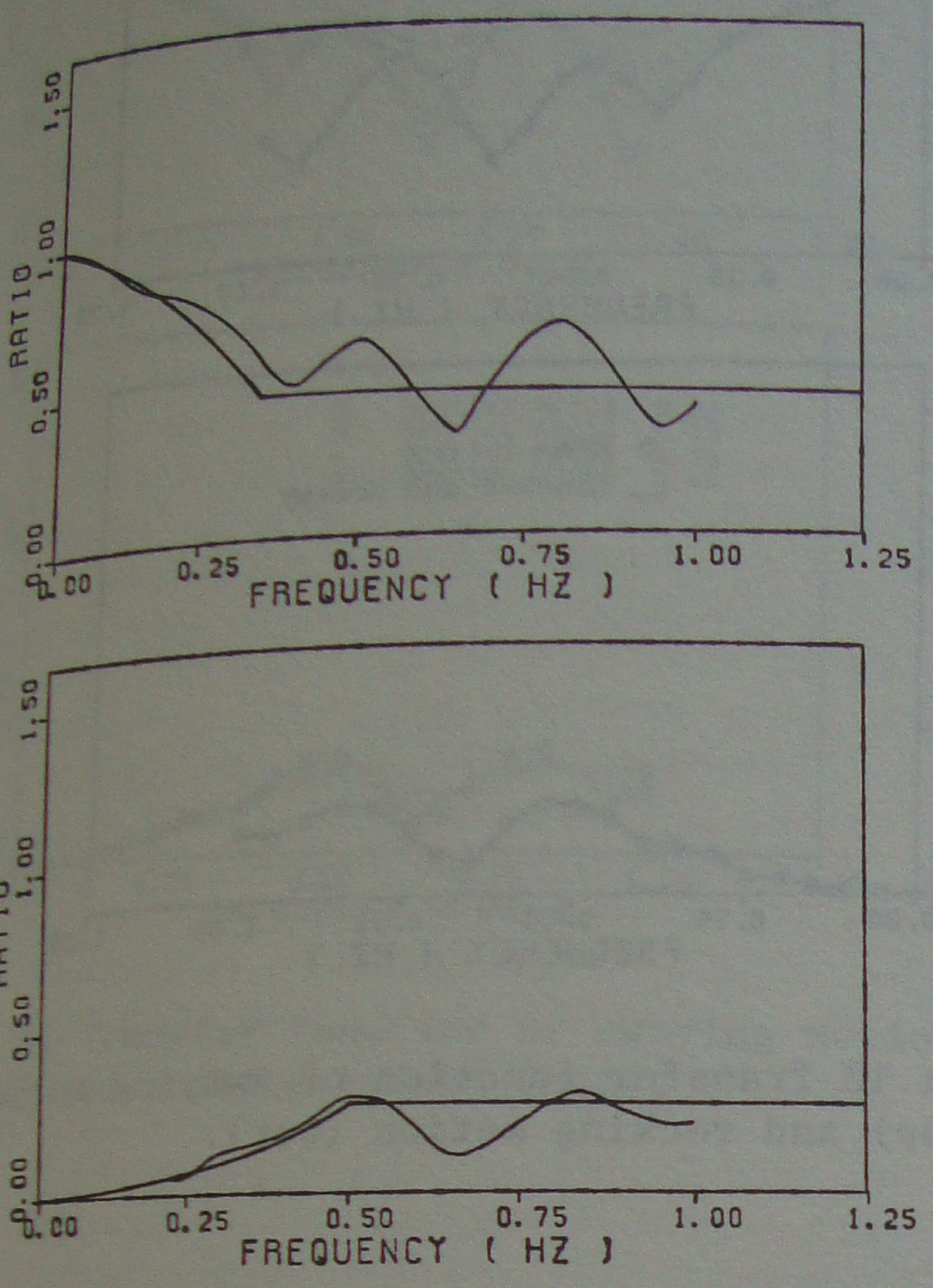

Fig.8 Transfer function of swaying motion (top) and rocking motion (bot).

Additional and more extensive studies on foundation motions (both surface and embedded) under trains of body waves at different angles of incidence as well as surface waves have been conducted by Iguchi (1973), Scanlon (1976), Luco (1976), Dominguez (1978), Pais and Kausel (1985) and Luco and Wong (1986) among others. The results show in all cases coupled reductions in the translational motions and occurrence of rotational components (both rocking and torsional terms for travelling waves) with a reduction of some effects as the others increase.

5 EFFECTS OF FOUNDATION EMBEDMENT

The studies by Elsabee and Morray (1977) had been conducted for a cylindrical foundation embedded in a homogeneous soil layer of finite depth resting on much stiffer rock. The embedment ratios E/R where E is the embedment depth and R the radius of the foundation varied from 0.5 to 1.5, a typical range for nuclear power plants. The value of Poisson's ratio for the soil was in all cases 0.33 and the internal soil damping, of a hysteretic nature, 0.05. These studies were extended by Kim (1984) considering different values of Poisson's ratio, soil damping and embedment ratios. The soil and the foundation were modelled as shown in figure 9 using three substructures: a rigid massless circular foundation (A), an assembly of soil finite elements under the foundation (B) and a lateral boundary (C) representing the far field (D). The lateral boundary was reproduced by the consistent boundary matrix developed by Kausel (1974) for the three dimensional case in cylindrical coordinates, which reproduces an extension of the finite element mesh from the foundation boundary to infinity. The foundation was idealized with zero mass and a stiffness equal to 10^8 times that of the soil. The dynamic stiffness matrix of the foundation and the underlying soil was assembled for each frequency and condensed to the degrees of freedom along the boundary imposing the condition of zero vertical displacements along the axis of the cylindrical region. Calling S_b the condensed dynamic stiffness matrix, R the boundary matrix, U_b the vector of displacements along the boundary and U_o P_o the displacements and nodal forces caused by the seismic waves along the boundary in the free field (forces acting on the far field) the resulting equilibrium equation is

$$(S_b + R)U_b = BU_o - P_o \qquad (41)$$

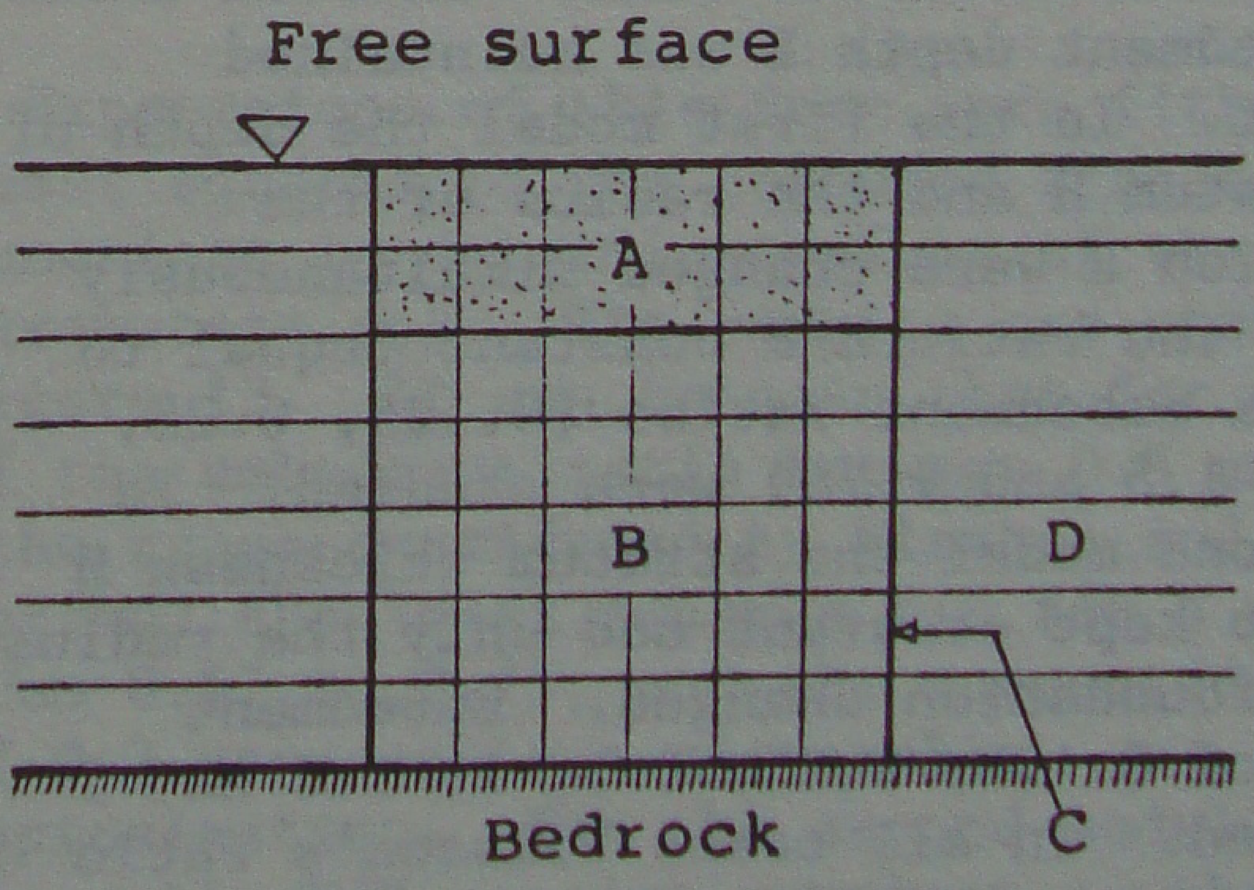

Fig.9 Model substructures.

From the solution of this equation one can obtain the horizontal and vertical displacement of the corner node of the foundation. Due to the very high stiffness of the finite elements reproducing the foundation the horizontal displacements are practically constant along the bottom of the foundation while the vertical displacements vary linearly with zero value at the center and a maximum at the edge. The vertical displacement of the corner divided by the radius represents therefore the rotation of the foundation. To assess the validity of the formulation and the computer program a number of cases identical to those studied by Elsabee and Morray (1977) were solved first and the results compared, with excellent agreement (see figure 10 for typical results). The effect of Poisson's ratio on the transfer functions was investigated next. Figure 11 shows the results for an embedment ratio of 0.5, a thickness of the soil layer H equal to twice the radius, 0.05 damping in the soil and Poisson's ratios of 0.33, 0.4 and 0.49. It can be seen that Poisson's ratio has very little effect within this range on either of the transfer functions. The effect of internal damping of the soil is illustrated in figure 12 for the same foundation and a soil with a Poisson's ratio of 0.33 and damping ratios of 0.02, 0.05, 0.10 and 0.15. As the damping ratio increases the transfer functions become smoother but their overall shape does not change significantly. The reduction in the amplitude of the translational motion for frequencies higher than the fundamental frequency of the embedment layer (or 70% of this frequency) decreases, however, as the damping increases.

To investigate the effect of the embedment ratio E/R on the results two different models were used. In both cases the embedment depth E was maintained constant. In the first model the depth of the stratum H and the radius of the foundation R were changed simultaneously keeping the ratio H/R constant (equal to 2). Six embedment ratios (0.125, 0.25, 0.5, 1, 1.5 and 1.75) were studied. In the second model the stratum thickness H was also kept constant and only the radius of the foundation changed. Embedment ratios of 0.125, 0.25, 0.5, 1, 1.5 and 2 were used. In all cases Poisson's ratio of the soil was 0.33 and the soil internal damping was 0.05. For each of the twelve cases studied the transfer functions were obtained as a function of a dimensionless frequency $a_o = \Omega R/c_s$ where Ω is the frequency of the motion in radions/second and c_s is the shear wave velocity of the soil. Elsabee and Morray (1977) had

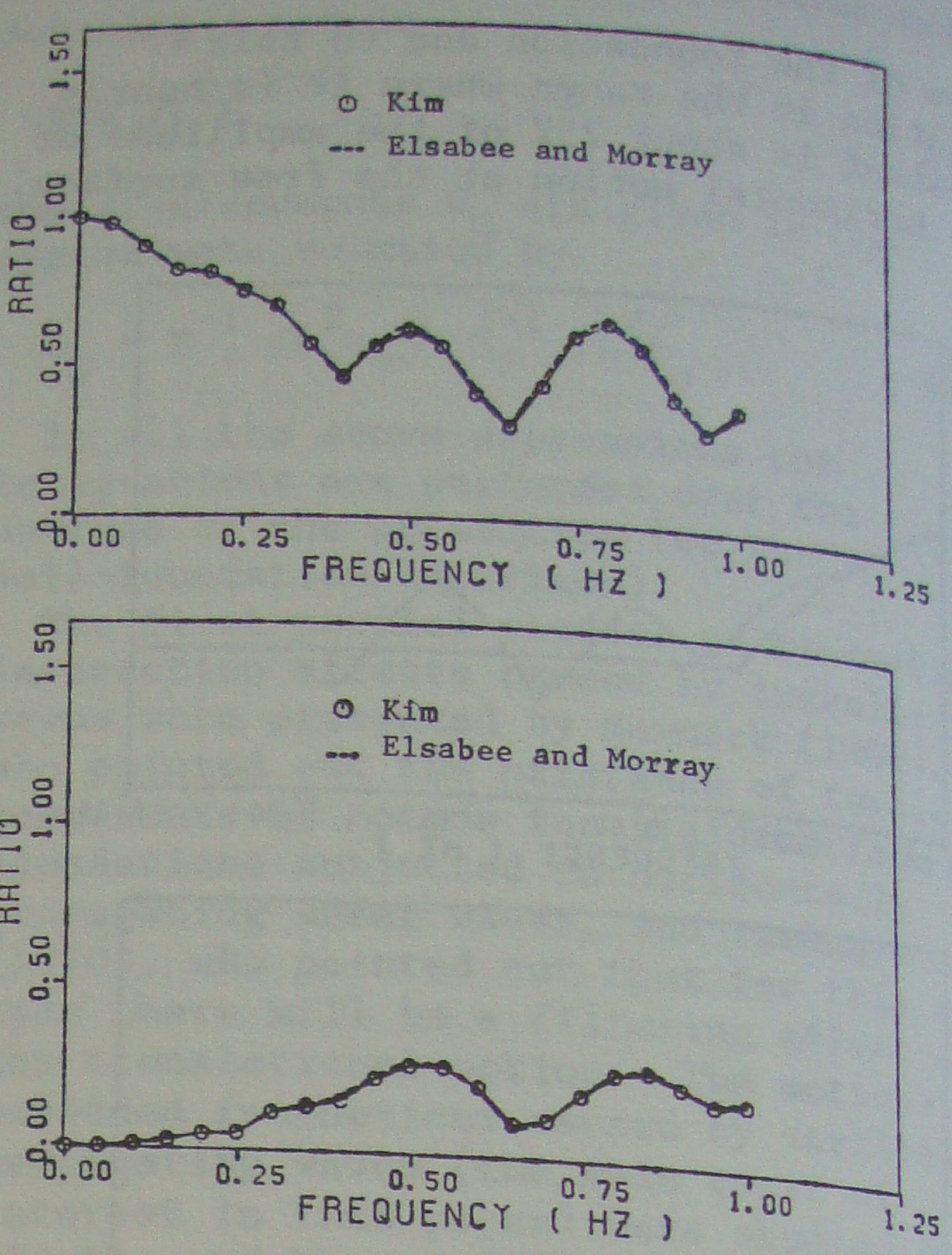

Fig.10 Transfer function of swaying motion (top) and rocking motion (bot).

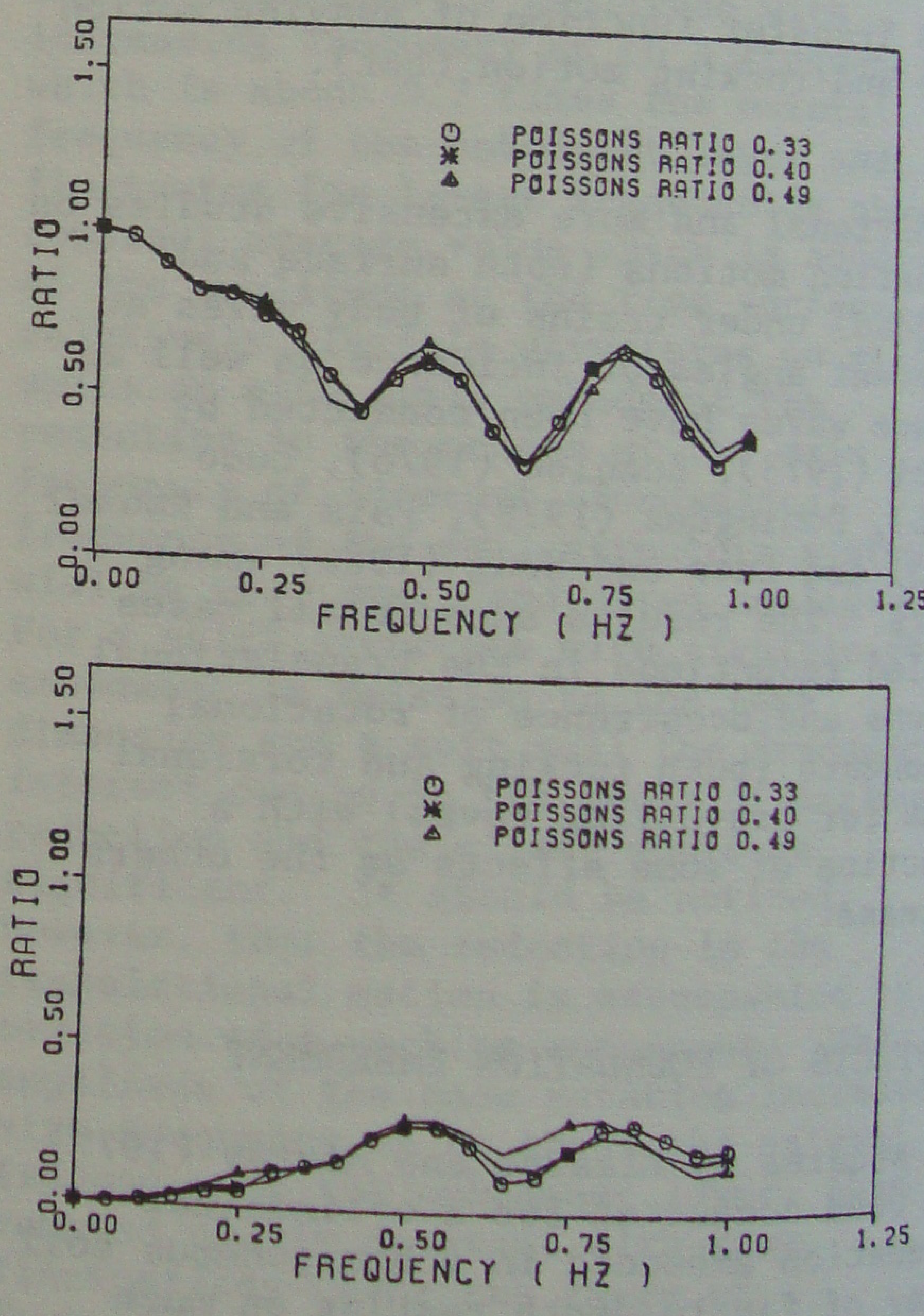

Fig.11 Transfer function of swaying motion (top) and rocking motion (bot).

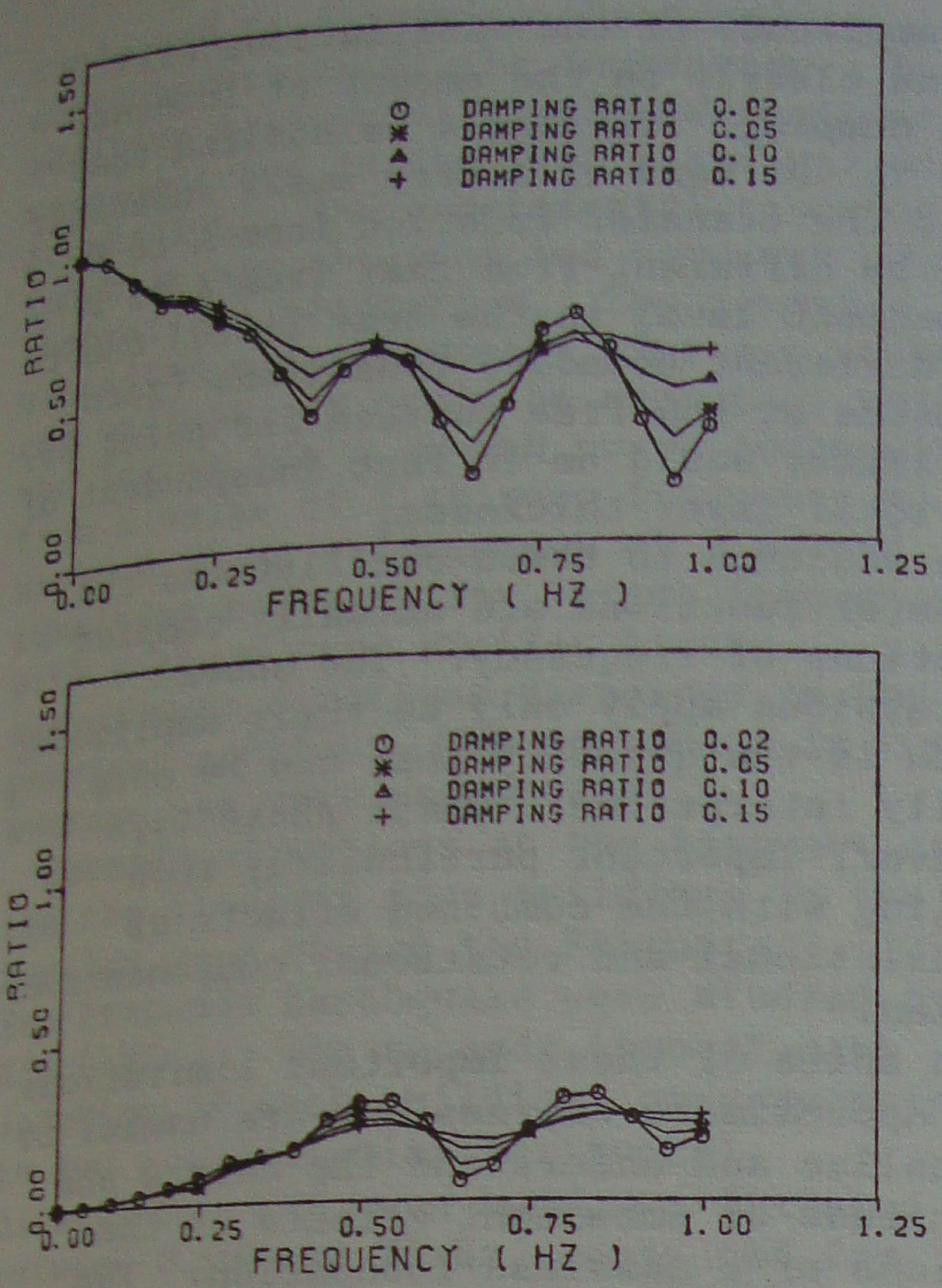

Fig.12 Transfer function of swaying motion (top) and rocking motion (bot).

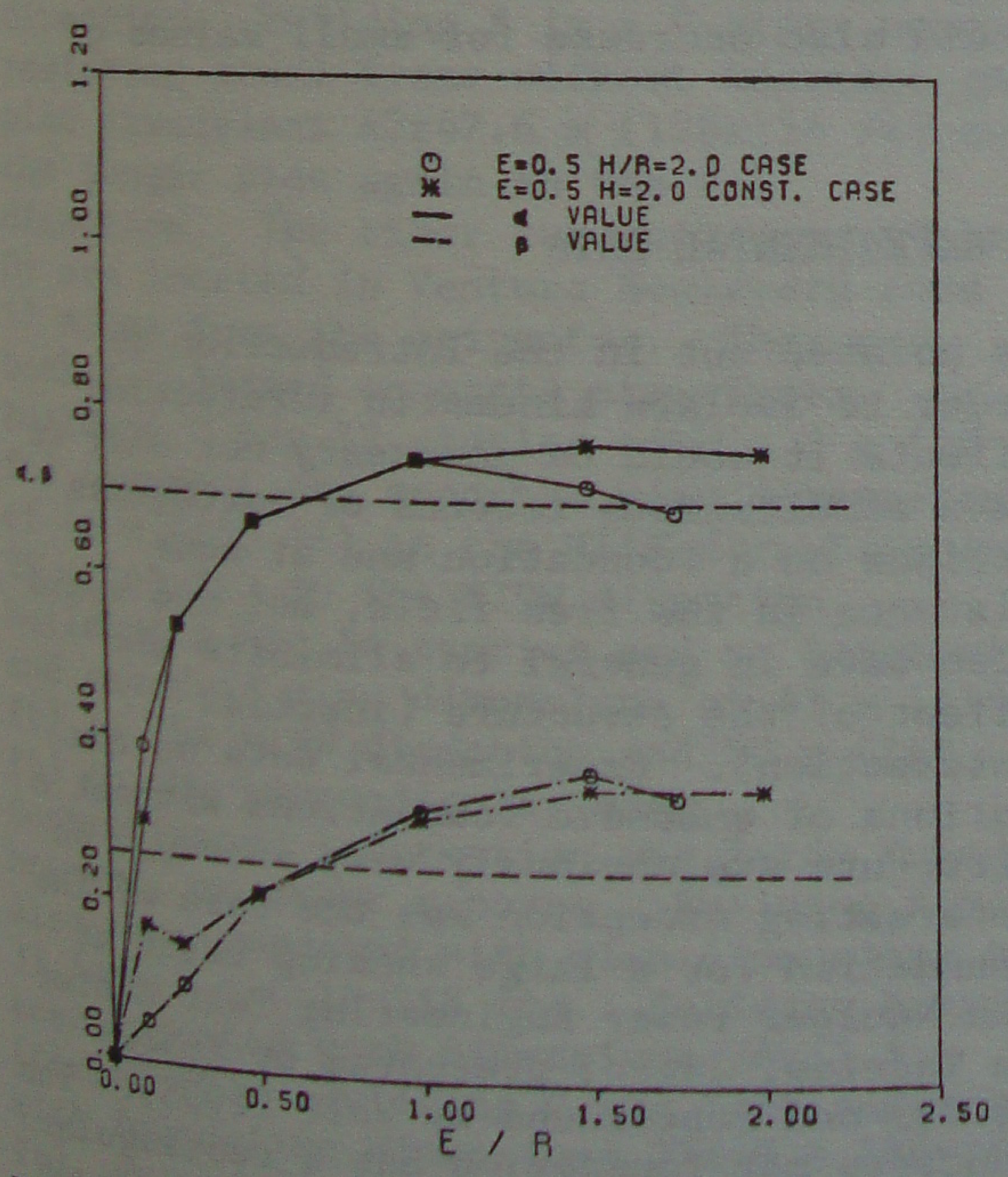

Fig.13 Least squares fitted α and β vs. E/R.

suggested approximate formulae for the translation and rotation at the base of an embedded foundation, relative to the amplitude of the translation at the free surface of the soil of the form

$$TF(U) = \begin{cases} \cos \frac{\pi}{2} \frac{f}{f_o} & \text{for } f \leq 0.7 f_o \\ \cos \frac{\pi}{2} (0.7 \frac{f}{f_o}) & \text{for } f \geq 0.7 f_o \end{cases} \qquad (42)$$

$$TF(\phi R) = \begin{cases} 0.257 \left(1 - \cos \frac{\pi}{2} \frac{f}{f_o}\right) & \text{for } f \leq f_o \\ 0.257 & \text{for } f \geq f_o \end{cases} \qquad (43)$$

where f_o is the fundamental frequency of the embedment layer

$$f_o = c_s/4E \qquad (44)$$

These expressions are independent of E/R. To introduce this variable they were generalized and written in the form

$$TF(U) = \begin{cases} \cos \frac{\pi}{2} \frac{f}{f_o} & \text{for } f \leq \alpha f_o \\ \cos \frac{\pi}{2} \alpha & \text{for } f \geq \alpha f_o \end{cases} \qquad (45)$$

$$TF(\phi R) = \begin{cases} \beta \left(1 - \cos \frac{\pi}{2} \frac{f}{f_o}\right) & \text{for } f \leq f_o \\ \beta & \text{for } f \geq f_o \end{cases} \qquad (46)$$

Expressions of this form were fitted by least squares to the computed transfer functions to determine values of the two parameters α,β. The results are shown in figure 13. It can be seen that for values of the embedment ratio between 0.5 and 1.5 (the range considered by Elsabee and Morray) the least squares fit for α varies from 0.67 to 0.75, and the suggested value of 0.7 represents a reasonable approximation. For the second model where the ratio E/H remains constant as the radius increases the value of α is essentially constant for E/R larger than 1. On the other hand when H/R is kept constant (first model) the value of α decreases slightly for values of E/R larger than 1. For embedment ratios less than 0.5 the value of α is considerably smaller than 0.7 varying from 0.3 for E/R = 0.125 to 0.54 for E/R = 0.25. It should be noticed that the amplitude of the transfer function for the translational motion provided by the approximate horizontal line for frequencies larger than αf_o is cos (πα/2). As the value of α decreases this amplitude approaches 1 indicating a much smaller average reduction in motion.

The value of 0.257 suggested by Elsabee and Morray (1977) for the parameter β is also reasonable for embedment ratios between 0.5 and 1. It is, however, much smaller for smaller values of E/R and larger as the embedment ratio increases. Again for the second model (constant value of E/H) the value of β is essentially constant for embedment ratios larger than 1, while it decreases slightly for values of E/R larger than 1.5 when H/R is kept constant.

It is important to notice that expressions (42), (43) originally suggested by Elsabee and Morray (1977) or their generalized forms (45), (46) recommended by Kim (1984) are only approximations intended to provide a feeling for the overall, or average, effects of embedment on the foundation motions (kinematic interaction effects). The true transfer functions for a particular situation will exhibit oscillations around these average curves. These oscillations are particularly pronounced for small values of the embedment ratio and the model where the stratum thickness remains constant. For a fixed embedment depth as the radius of the foundation increases the transfer function from point A to point C in figure 1, tends to become the ratio of the amplification function for a soil deposit with a thickness H-E to the amplification function for a stratum with thickness H. If the soil properties are homogeneous these two amplification functions would have exactly the same shape on a different frequency scale: each one would exhibit peaks at the corresponding natural frequencies of the stratum. As the depth of the soil stratum increases approaching a half space the ratio of the amplification functions tends to one over the complete frequency range. On the other hand for a stratum of finite depth and particularly for a shallow one (relative to the embedment depth) the ratio will exhibit marked valleys at the natural frequencies of the soil deposit and peaks at the frequencies of the soil layer below the foundation (of depth H-E). The amplitude of the oscillations will depend clearly on the amount of internal soil damping. It should be noticed that even in the case of a very small embedment ratio the transfer function from A to C will be different from that from A to B (embedment level in the free field) due to the different boundary conditions (zero stresses at the free surface for point C). The latter would be in fact independent of the total layer thickness.

In addition to these oscillations the transfer functions are actually complex functions of frequency. The above expressions apply only to their amplitude, which is the quantity that can be more easily interpreted. Their phase is, however, important particularly when dealing with the combined effects of translational and rotational components of motion.

In spite of these important limitations the approximate expressions are useful to visualize and understand the nature and magnitude of embedment effects on the motions of a massless foundation. The rules suggested by Elsabee and Morray (1977) are reasonable for embedment ratios of the order of 0.5 or larger, often encountered in nuclear power plants (the range that had been considered in their study). For smaller embedment ratios the reduction of the translational motion with depth is much smaller and use of expressions (42), (43) would produce unconservative estimates. On the other hand the rotation induced at the base would also decrease for small values of E/R.

6 EXPERIMENTAL DATA

As pointed out in the Introduction in order to isolate kinematic interaction effects it would be necessary not only to have simultaneous records of earthquake motions on a foundation and at some distance in the free field, but one would also have in general to eliminate the effect of the structure (inertial interaction). Experimental data on the motions of embedded foundations without a structure are obviously very scarce. An interesting exception was the case of the foundation for a large shaking table at the Nuclear Power Engineering Test Center in Tadotsu, Japan, presented by Tajimi (1983) and reproduced by Wolf (1985). The concrete mat foundation has a rectangular plan 91x45 m (298x147 ft), a thickness of 21 m (69 ft) at the center and 13 m (43 ft) at the edge. Vibration tests were conducted to determine the dynamic characteristics of the foundation. In addition horizontal earthquake motions were recorded in the free field and on the foundation. The experimental transfer function was obtained and compared to a theoretical prediction. The former exhibited a large number of sharp peaks and valleys, as could be expected, while the latter was a smooth curve decreasing from a value of 1 at zero frequency to about 0.35 at a frequency of 3 Hz and remaining constant for larger frequencies, with a shape very similar to the approximate functions discussed in the previous section. The agreement in the average was very reasonable.

Records of motions at the basement and the top floor of four buildings in the Los Angeles area during the February 9, 1971 San Fernando Earthquake were studied by Kim (1984). The records (two at each location of each building, corresponding to two perpendicular horizontal directions) had been bandpass-filtered between 0.07 and 25 Hz, corrected for instrument and baseline, and digitized at 0.02 seconds intervals. They were provided by Woodward Clyde Consultants.

Two of the buildings (A and B) were located nearby in Wilshire Boulevard and West 6th Street respectively, about 24 miles from the epicenter. Building A is a 12 story reinforced concrete structure with a 3 m (10 ft) basement. Its base dimensions were reported as 34x78 m (111x257 ft) with the long side on the N-S direction. Building B is a 9 story moment resisting steel frame without basement and plan dimensions 42x47.6 m (138x156 ft) and the longer side again in the N-S direction. The other two buildings (C and D) are located in Ventura Boulevard some 17 miles from the epicenter. They are both reinforced concrete structures. Building C has 12 stories with a 3 m (10 ft) basement and reported plan dimensions of 46 m (150 ft) in the N11E direction and 20 m (65 ft) in the N79W direction. Building D had 14 stories and no basement and reported plan dimensions of 41 m (135 ft) in the N12W direction and 50 m (165 ft) in the perpendicular one. Contradictory information was later found, however, in other reports. Building A has also been reported with plan dimensions of 33.5x66.5 m (110x218 ft). Building B has been reported with dimensions 26x43.5 m (86x143 ft). Building C has been reported with foundation dimensions 27x49 m (90x161 ft), ground floor dimensions 29.5x53 m (97x175 ft) and plan dimensions for the third to top floors of 18x49 m (60x161 ft). Building D had an irregularly shaped foundation with overall dimensions 37x54.5 m (121x179.5 ft) and typical floor dimensions 23x47.5 m (76x156 ft). Of primary importance for the analytical part of the study, to determine kinematic interaction effects, are the foundation dimensions for buildings A and C that have a basement where motions were recorded. The variations in the dimensions reported for building A are small, those of building C are larger but do not affect significantly the main conclusions. Buildings B and D had no basements and it was initially assumed, as a first approximation, that the motions recorded at their base would be similar to those that would have occurred on the free surface of the soil in the free field. Inertial interaction effects due to the structures were eliminated later in an approximate way. Building D was found to have, however, a pile foundation that would affect somewhat the motions. As a consequence the results based on the analysis of buildings C and D are less reliable than those obtained from buildings A and B.

The soil properties in the general areas of the buildings were furnished by Woodward Clyde Consultants. A deconvolution process was applied to determine values of shear moduli (or shear wave velocities) and damping consistent with the levels of strains induced by the recorded earthquakes. The resulting properties (thickness of the soil layers h, shear wave velocity c_s and damping ratio D) are shown in table 1. They differ very little from the original ones. Site 1 is the one corresponding to buildings A, B while site 2 is that of buildings C, D.

A first estimate of the effect of embedment on foundation motions (kinematic interaction) can be obtained, neglecting inertial interaction effects, by dividing the Fourier transforms of the recorded motions at the basements of buildings A and C by those of the records obtained at the base of buildings B and D respectively. This yields the transfer functions from the base of buildings B and D to the base of buildings A and C. These transfer functions are extremely jagged with a large number of sharp peaks and valleys, which make it difficult to visualize their basic trends. Two different approaches were used to extract overall trends from this information. The first one was to smooth the amplitude of the transfer functions using average values over different frequency windows maintaining the area under the curves. Figure 14 shows the results for the transfer function from the base of building B to building A in the N-S direction using averaging windows with 16 points (a frequency range of 0.2 Hz), 32, 64 and 256 points (a frequency window of about 3 Hz). It can be seen that with the largest window a smooth curve very similar in overall shape to those discussed earlier is obtained. The second approach was to fit to the transfer functions (the original ones, not the smoothed ones) expression of the type of equation (45). The results using this approach are shown in figures 15 and 16. The least squares fit was performed using for f_o the natural frequencies of the embedment layers computed from the soil properties in table 1. The corresponding values were 27.5 Hz for site 1 and 17.5 Hz for site 2.

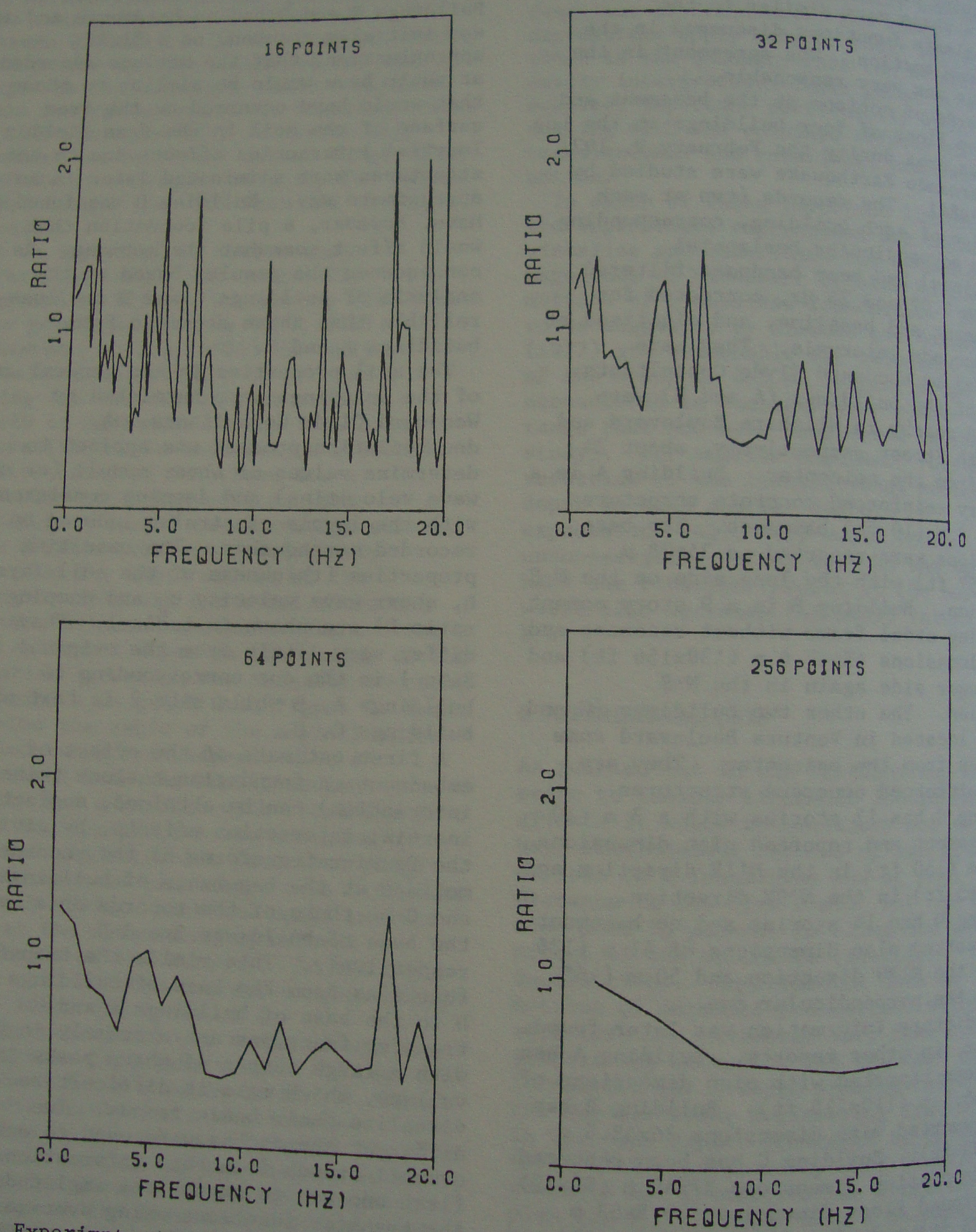

Fig.14 Experimnt. smoothed transfer function N00E (R1/R5) Wilshire Blvd. area.

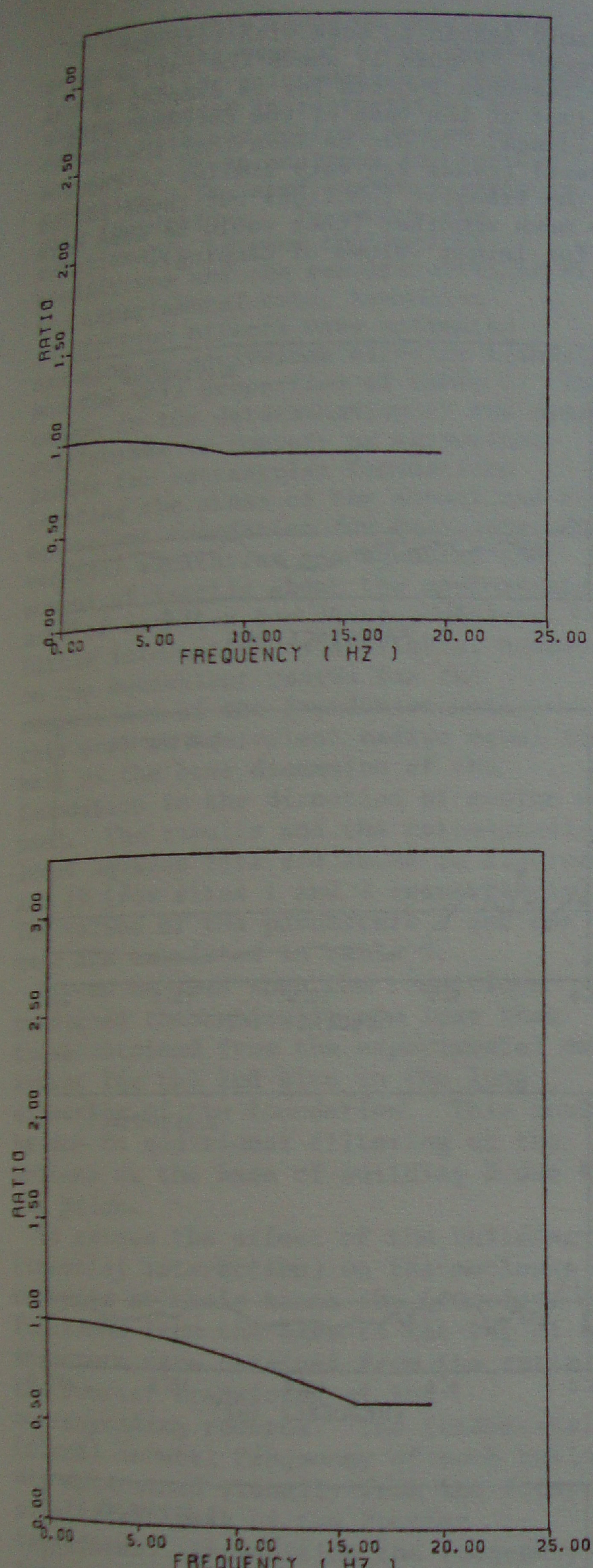

Fig.15 Transfer function of swaying motion W.R.T F.S. 3470 Wilshire Blvd.

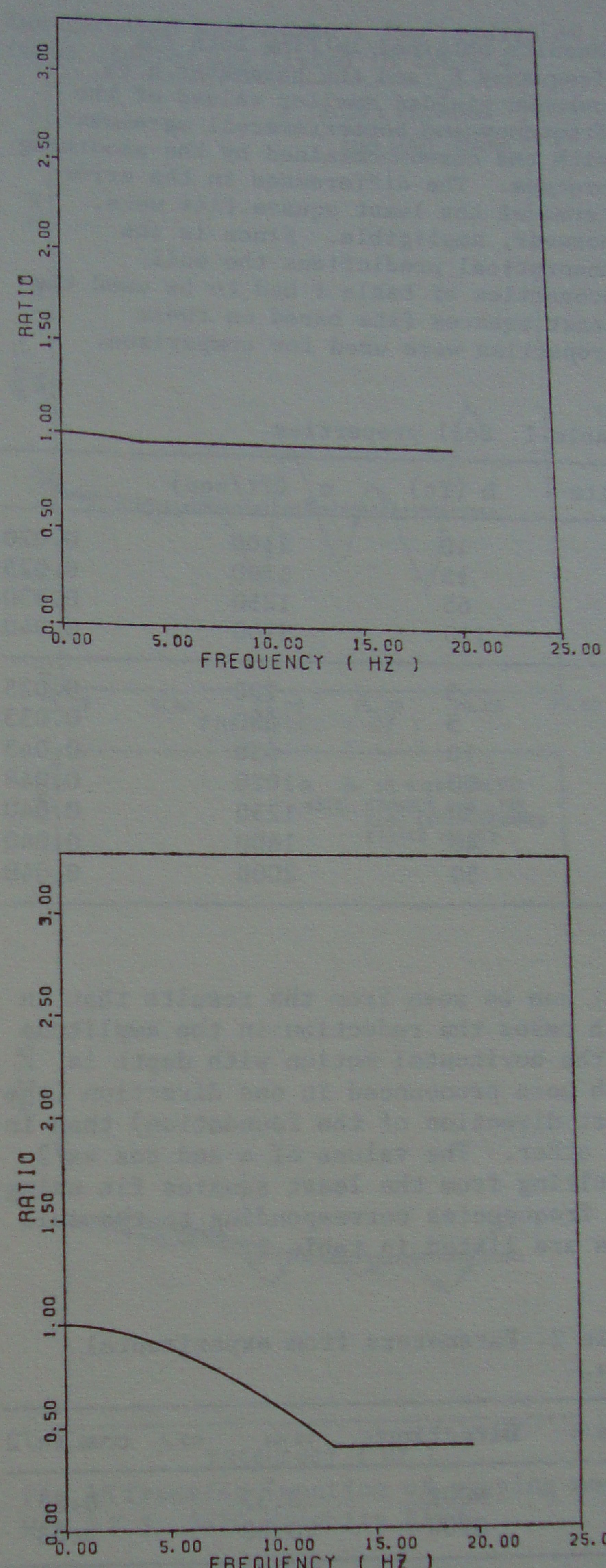

Fig.16 Transfer function of swaying motion W.R.T F.S. 15250 Ventura Blvd.

Results obtained letting both the frequency f_o and the parameter α be unknown yielded smaller values of the frequency and better overall agreement with the curves obtained by the smoothing process. The differences in the error terms of the least square fits were, however, negligible. Since in the theoretical predictions the soil properties of table 1 had to be used the least squares fits based on these properties were used for comparison.

Table 1. Soil properties.

Site	h (ft)	c_s (ft/sec)	D
1	10	1100	0.020
	15	1100	0.025
	65	1250	0.030
	110	2100	0.040
2	5	700	0.025
	5	680	0.033
	10	650	0.043
	60	1020	0.048
	30	1250	0.040
	40	1600	0.040
	50	2000	0.040

It can be seen from the results that in both cases the reduction in the amplitude of the horizontal motion with depth is much more pronounced in one direction (the short direction of the foundation) than in the other. The values of α and cos πα/2 resulting from the least squares fit using the frequencies corresponding to the soil data are listed in table 2.

Table 2. Parameters from experimental data.

Site	Direction	α	cos πα/2
1	N00E	0.33	0.87
	S90W	0.58	0.63
2	N11E	0.20	0.95
	N79W	0.74	0.40

While it is convenient to use the ratio of the Fourier transforms of the motions to compare the experimental transfer functions to analytical predictions (independent of the characteristics of the earthquake motions) it is more usual in seismic design to work with response spectra. Figure 17 shows the ratios of the response spectra for 5% damping of the motions at the base of the corresponding buildings. It can be seen that the general trends are very similar to those of the transfer functions but these ratios are much smoother (they would be even more so for larger values of damping).

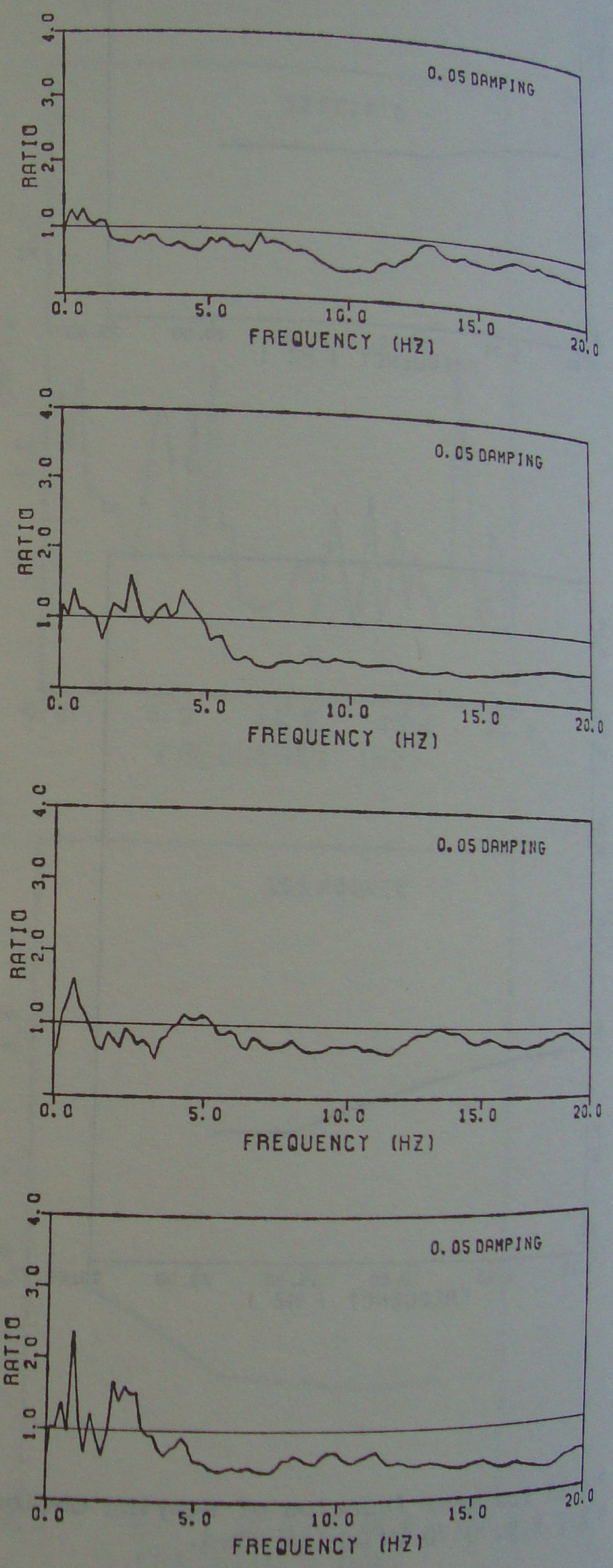

Fig.17 Ratio of response spectra.

It is possible today to obtain the motions at the base of a rectangular foundation (or a foundation of arbitrary shape) embedded in a stratified soil deposit with any desired degree of accuracy. These analyses are however expensive. The same comments apply to pile foundations. To assess the relationship between theoretical predictions and the results obtained from the experimental data, kinematic interaction effects were estimated assuming an equivalent circular foundation and the soil properties of table 1. It is common in the determination of the dynamic stiffnesses to compute an equivalent radius for rectangular foundations equating the areas of the actual and the equivalent foundation for horizontal and vertical excitation and equating the moment of inertia about the appropriate axis of rotation for rocking motions. Little information is available, however, on the equivalent radius for the computation of the foundation motions. In this work an equivalent radius equal to half of the base dimension of the foundation in the direction of motion was used. The results and the corresponding least squares fits are shown in figures 18 and 19 (for sites 1 and 2 respectively). The values of the parameters α and cos πα/2 are tabulated in table 3.

It can be seen that the reductions predicted theoretically are less than those obtained from the experimental data except for the 2nd site in the long direction of the foundation. This could be due to additional filtering of the motions at the base of building D due to the piles.

To assess the effect of the buildings (inertial interaction) on the motions recorded at their bases the transfer functions from the base to the top of each structure were obtained from the ratio of the Fourier transforms of the corresponding records. The fundamental (first) natural frequency of each building was determined visually from the first significant peak of the Fourier transforms. In addition the frequency and the corresponding damping were also computed using a least squares fit and assuming for each building an equivalent shear beam. It should be noticed that the values of the natural frequency are those corresponding to the behavior of the building during the earthquake. These values agree well with those reported in the literature for the buildings for which dynamic analyses were performed after the San Fernando earthquake. The results of these studies are listed in table 4.

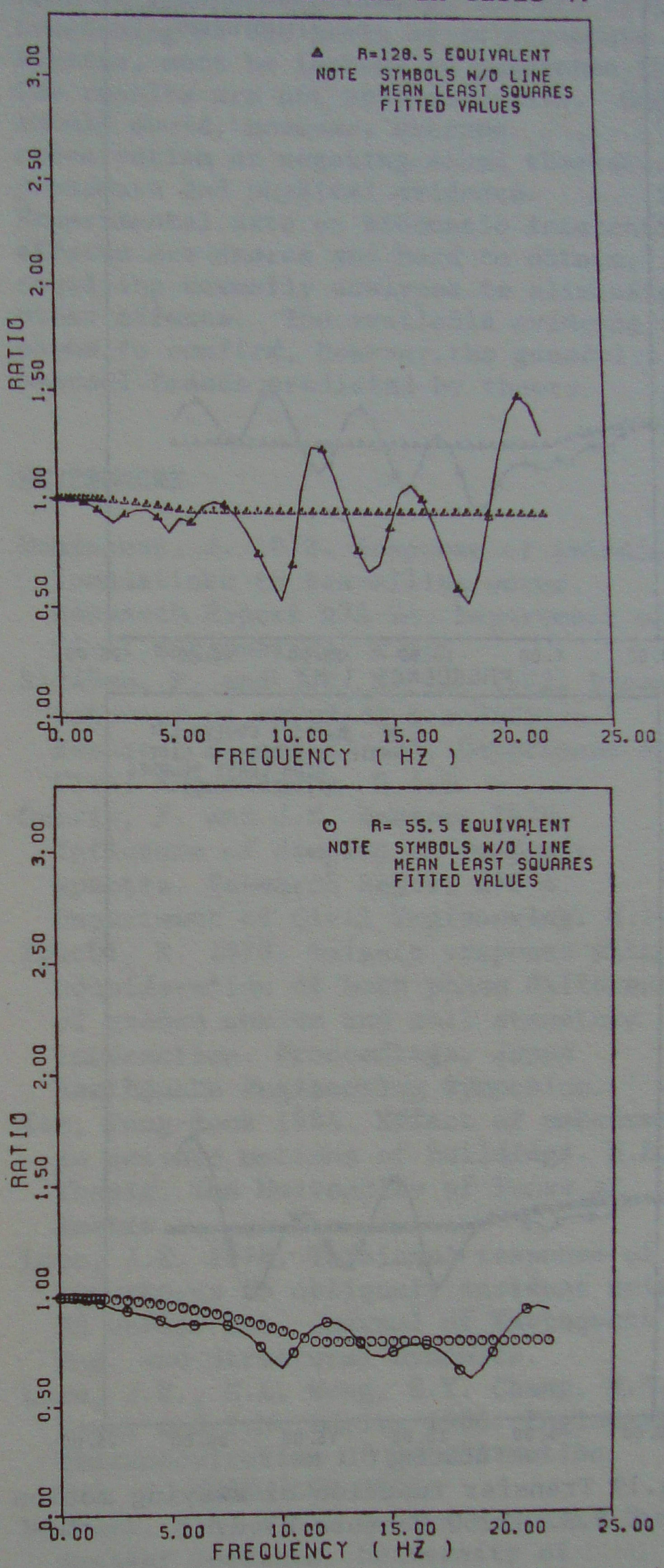

Fig.18 Transfer function of swaying motion W.R.T F.S. 3470 Wilshire Blvd.

Table 3. Theoretical predictions without inertial interaction.

Site	Direction	α	cos α/2
1	N00E	0.21	0.95
	S90W	0.42	0.79
2	N11E	0.29	0.90
	N79W	0.52	0.68

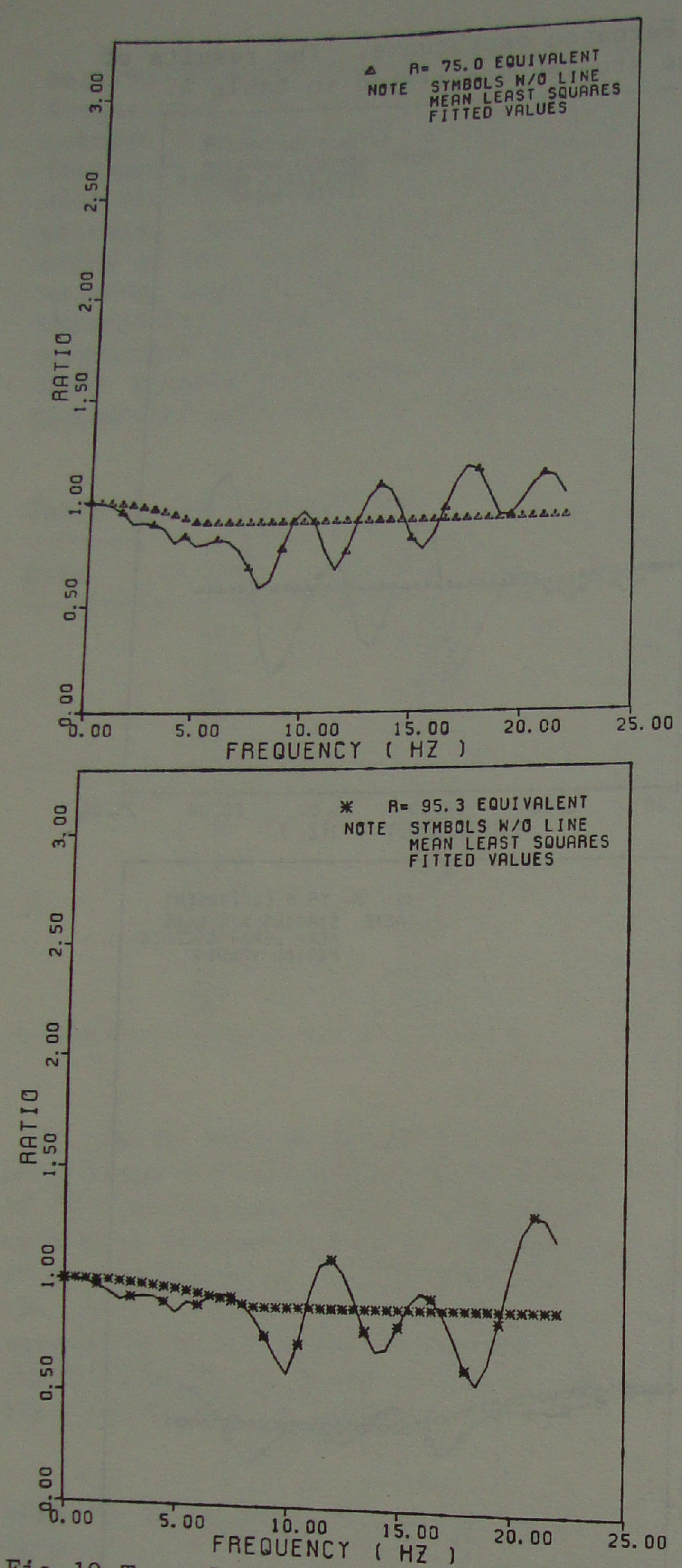

Fig.19 Transfer function of swaying motion W.R.T F.S. 15250 Ventura Blvd.

Table 4. Equivalent building properties.

Building	Direction	Least Squares f_o	D	Peak f_o
A	N00E	0.63	0.03	0.635
	S90W	0.68	0.02	0.684
B	N00E	0.61	0.03	0.610
	S90W	0.60	0.02	0.600
C	N11E	0.51	0.03	0.513
	N79W	0.33	0.04	0.330
D	N12E	0.96	0.02	0.952
	N78W	0.89	0.03	0.891

Using equivalent shear beams with the cross sectional area of the foundations, estimated heights based on the number of stories and an equivalent mass density of 290 kg/m³ (18 lbs sec²/ft⁴), and selecting the shear modulus so as to obtain the desired natural frequency, the ratios of the motions at the buildings A and B, and C and D, were computed including both kinematic and inertial interaction effects. Least square fits were obtained again for these new results. The values of the parameters α and cos πα/2 for these cases are listed in table 5. Comparing these results to those of table 3 it can be seen that inertial interaction effects cause an increase in the values of α and therefore a further reduction of the motions at the base of the buildings, although the change is very small in the present case. The agreement with the experimental data (which included the inertial interaction effects) improves for the three cases where the theoretical predictions had yielded smaller reductions and deteriorates for site 2 and the long direction of building C.

Table 5. Theoretical predictions with inertial interaction.

Site	Direction	α	cos πα/2
1	N00E	0.25	0.92
	S90W	0.45	0.76
2	N11E	0.32	0.88
	N79W	0.57	0.62

While the agreement between the theoretical predictions and the experimental data is far from perfect, when considering the many uncertainties in the soil and structural properties and the simplifications introduced in the mathematical models (some of them rather crude), the general trends agree reasonably well. The fact that the amplitude of the horizontal motions will decrease with embedment depth, particularly above a certain frequency, as predicted by theoretical studies, seems to be clearly illustrated not only by the smoothed experimental transfer functions but also by the ratio of the response spectra. It should also be noticed that in three of the four cases the experimental results suggest larger reductions than those predicted. The exception is the case where the reference surface building had a pile foundation with a large number of piles in that direction, which were not accounted for in the analysis.

7 SUMMARY AND CONCLUSIONS

There has been in recent years some controversy as to the location where the control motion for the seismic analysis of structures with embedded foundations should be specified. From a physical point of view the recommendation adopted for some time that the design earthquake be specified at the foundation level in the free field is the worst possible alternative. The seismic input should be specified at the free surface of the soil when it is defined in terms of generic R.G. 1.60 type spectra for average soil conditions, as a collection of spectra for various soil categories (rock, firm ground, deep soil deposits) or as actual records corresponding to the same general conditions (magnitude, distance, soil properties). For a direct soil structure interaction analysis where the structure and the soil are modelled together compatible motions (and stresses) should be computed at the bottom and lateral boundaries of the model. For a substructure or three step approach the input to the inertial interaction analysis should be the motions at the base of a massless foundation resulting from kinematic interaction analyses. In both cases an assumption has to be made as to the types of waves. In the absence of more detailed information the assumption of vertically propagating waves is a reasonable one. For a substructure approach the foundation motions for a rigid foundation should include rotational components. In addition some provision should be made for torsional motions in both approaches. For generic R.G. 1.60 spectra an additional eccentricity as normally assumed seems to provide sensible results. For site specific motions larger eccentricities may be required.

The only other logical alternative is to specify the input in terms of a rock spectrum at rock outcropping. One would have to compute again compatible motions in the free field and the same comments made above will apply. In all cases the soil properties and layering should be modelled as accurately as possible within the available data.

Uncertainties will exist not only in soil structure interaction analyses but in all phases of seismic design, including the modelling of the structure, and reasonable provisions, based on judgment, such as smoothing of design spectra or broadening of the peaks of in structure spectra, must be imposed to guarantee that the results are not unconservative. One should avoid, however, extreme conservatism or negating sound theoretical procedure and physical evidence. Experimental data on kinematic interaction effects are scarce and hard to obtain, requiring normally analyses to eliminate other effects. The available evidence seems to confirm, however,the general overall trends predicted by theory.

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