GEOTECHNICAL ENGINEERING

SASSI

A SYSTEM FOR ANALYSIS OF

SOIL-STRUCTURE INTERACTION

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A SYSTEM FOR ANALYSIS OF SOIL-STRUCTURE INTERACTION

by

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1. INTRODUCTION

The currently available methods of soil-structure interaction analysis may be subdivided into two main groups: continuum methods and finite element methods.

The continuum methods can properly handle three-dimensional effects but their use is for practical purposes limited to the case of rigid foundations. The finite element methods can handle flexible foundations but are usually applicable only to plane strain or axisymmetric situations. Since embedment is known to be an important factor and since most engineering structures are truly three-dimensional, there is clearly a need to develop more general methods of analysis.

It is therefore proposed to develop a System for Analysis of Soil-Structure Interaction (SASSI) which can handle much more complex geometries and seismic environments than current methods of analysis. A pilot study of such a system is currently in progress at the University of California at Berkeley, California, USA. The study has shown that it is possible to develop an efficient system with the following capabilities and limitations:

a) The site is horizontally layered. (visco-elastic layers over a visco-elastic half-space).

b) The seismic environment consists of an arbitrary three-dimensional superposition of, perhaps inclined, plane body waves (P- and S-waves) and surface waves (R- and L-waves).

c) The structure(s) can be represented by standard three-dimensional finite element models connected to the soil at several points within the embedded part of the structure.

d) The analysis is essentially linear, i.e., the equivalent linear method is used to approximate primary nonlinear effects in the free field. Secondary nonlinear effects may be considered in a limited region near the structure.

Within the above limitations and that of available computer capacity the system can handle embedded structures with flexible basements, structure-structure interaction, the effects of torsional ground motions, pile foundations, etc.

As will be discussed below, the system and the associated computer code, SASSI, are modular such that the individual parts of the analysis can be performed separately and complete reanalysis is not necessary if say the structure or the seismic environment are changed.

The system works in the frequency domain and is therefore applicable both to deterministic and probabilistic analysis.

In addition to seismic loads it is also possible to introduce external forces such as impact loads, wave forces or loads from rotating machinery acting directly on the structure.

The project has been going on since January 1979 and all of the key elements of the system have now been completed. The remaining tasks are: implementation of several features which will increase the efficiency and utility of the code, additional verification of the code by comparison with known solutions and application in case studies, development of a number of pre- and post-processors, manuals and other documentation.

2. SOIL-STRUCTURE INTERACTION

The current state-of-the-art of seismic soil-structure interaction (SSI) is described in Refs. 1 and 2.

Seismic loads will be discussed first. All seismic SSI problems involve two distinct parts: the site response problem and the interaction problem.

2.1 The Site Response Problem

The site response problem involves the determination of the temporal and spatial variation of the motions in the free field (prior to construction); usually from a single specified control motion at some control point. As discussed in Ref. 2, it is usually best to choose a control point at the ground surface or at an imaginary outcrop. For the system of analysis described herein it is sufficient to determine the free-field motions within the soil volumes to be excavated before placement of the structure(s).

Site response problems are ill-posed and unique solutions can be obtained only by the introduction of restrictive assumptions regarding the geometry of the site and the nature of the wave field causing the control motion. In practice, consistent solutions can be obtained only for horizontally layered sites. Hence, the proposed system is limited to this geometry. Possible wave patterns include: vertically propagating body waves, inclined body waves and horizontally propagating surface waves.

2.2 The Interaction Problem

The interaction problem involves the determination of the response of the structure(s) to be placed in the seismic environment determined from the site response analysis. As described in Ref. 2, this analysis can be performed in several ways each of which involves superposition of two fields of motion. Thus truly nonlinear analysis of SSI is not currently possible. However, nonlinear effects can be approximated by an equivalent linear method.

2.3 Substructure Methods

The proposed system employs a substructure method and only such methods will be discussed herein. At the first glance substructuring is the obvious way to handle SSI problems and a large number of methods have been proposed in which the soil mass is treated as a continuum and the structure as a discretized model. The half-space is analyzed first, usually in the frequency domain, and the impedance and scattering properties at the soil-structure interface are established. In the second step these properties are used as boundary conditions in a dynamic analysis of the structure with a loading which depends on the free-field motions. In recent years several substructure methods have appeared in which the half-space solution is obtained by finite element analysis with transmitting boundaries.

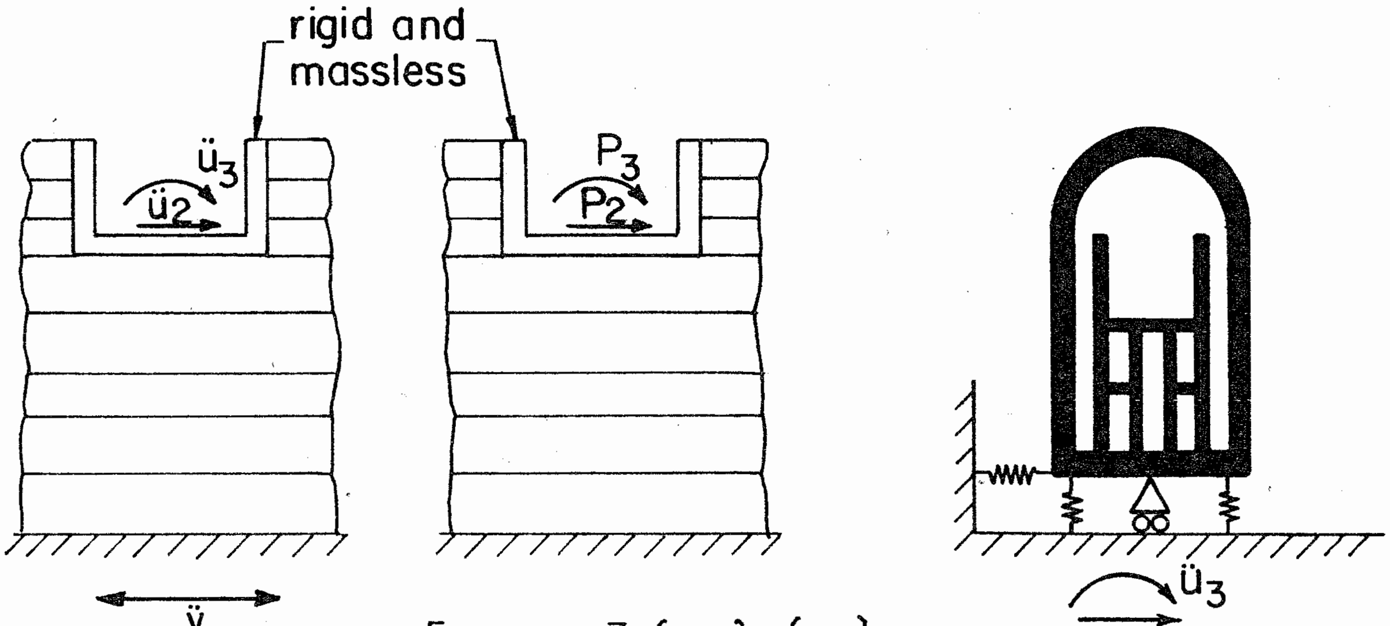
For the case of structures founded at the surface substructuring becomes extremely simple, especially for the case when the seismic environment consists of vertically propagating waves, see Ref. 2. However, most real structures are embedded into the half-space. The effects of even shallow embedment can be quite significant and simple substructuring methods cannot handle this problem. The basic difficulty in solving the embedded soil-structure interaction problem is that, because the free-field motions vary considerably with depth, especially in softer materials, it is difficult to specify the distribution of the free-field forces on the embedded part of the structure.

The substructure methods which can handle the embedment problem fall into three groups: rigid boundary, flexible boundary and flexible volume methods. This terminology refers to the region over which the structure and the soil are connected.

Rigid Boundary Methods

Kausel and Roesset (1974) have proposed a rigorous 3-step method for the case of rigid embedded foundations. This procedure is illustrated in Fig. 1. The first step is a scattering problem in which the site includes a rigid massless foundation with the same shape as the actual foundation. The solution to this problem produces a set of rigid body accelerations, $\ddot{u}_i$, for points in the structure. The second step is a foundation vibration (impedance) problem the solution to which is the impedance matrix for the foundation and with this the springs and dashpots to be used in the third step of the analysis where the loading on the structure is computed from the free-field motions obtained from Step 1. The total displacements follow by superposition of the motions from steps one and three. While the Kausel-Roesset method has been presented only for the case of vertically propagating shear waves it is in fact applicable with other wave fields. Unfortunately, rigorous solutions to the first step are difficult to obtain except by finite element methods and since the second step also requires a finite element analysis the complete 3-step method is rather costly. However, with approximate solutions to Step 1, Kausel et. al. (1978), and Step 2, Novak and Beredugo (1972), Kausel and Roesset (1975), the method does lead to economical analyses for the case of vertically propagating waves.

A different, but similar, formulation of the rigid foundation interaction problem has been proposed by Luco et. al. (1975) and used by Day (1977, 1978) to determine the response of structures on cylindrical embedded



$$\begin{bmatrix} K_{22} & K_{23} \\ K_{23} & K_{33} \end{bmatrix} \begin{Bmatrix} U_2 \\ U_3 \end{Bmatrix} = \begin{Bmatrix} P_2 \\ P_3 \end{Bmatrix}$$

(a) Scattering Problem

(b) Impedance Problem

(c) Structural Analysis

Fig. 1 3-STEP METHOD FOR INTERACTION ANALYSIS foundations subjected to inclined SH-waves. The method is similar to the 3-step method discussed above in that it usually requires two finite element analyses of the models Fig. 1(a) and Fig. 1(b). The first analysis (the scattering problem) involves determining the forces which the free-field motions exert on the footing when this is held fixed in space. The second analysis (the impedance problem) is identical to that of the above procedure. The third step, which involves matrix algebra is as simple to solve as Step 3 in Fig. 1(c). Structural engineers may recognize this method as a sophisticated version of the slope-deflection method. The method can, in principle, consider structure-soil-structure interaction.

A few continuum solutions have been obtained to the above scattering and impedance problems for embedded shapes in a perfect half-space. Thus, Thau and Umek (1973, 1974), Thau et. al. (1974), Dravinski and Thau (1976a, 1976b), Luco et. al. (1975), and Trifunac (1972) have developed solutions for a number of plane-strain problems involving rigid footings of rectangular and semi-circular cross sections. The most common excitation is SH-waves. Some three-dimensional problems involving rigid semi-spherical and semi-ellipsoidal foundations have been solved by continuum methods, Luco (1976b), Apsel and Luco (1976). Considering the mathematical difficulties in obtaining continuum solutions for more complicated, perhaps flexible, shapes embedded in layered systems and excited by different seismic environments, it must realistically be assumed that for practical problems both the scattering problem and the impedance problem have to be solved by some type of discretized procedure, say finite element methods.

Flexible Boundary Methods

If the flexibility of the embedded part of the structure is to be considered, a flexible boundary method must be used. Rigorous finite element methods for this case have been developed by Chopra and Gutierrez (1973) and Aydinoglu (1977) for surface structures and by Gutierrez (1976), Gutierrez and Chopra (1977, 1978) for embedded structures. These methods have, in principle, the ability to consider not only flexible foundations but also structure-soil-structure interaction.

As with the methods discussed above, the complete solution of a soil-structure interaction problem requires first the evaluation of a site response (scattering) problem similar to that shown in Fig. 1(a) to determine the motion of the now flexible boundary, and second the solution of an impedance problem similar to the one stated in Fig. 1(b). The latter problem now involves more degrees-of-freedom and leads to a larger impedance matrix. The third step involves a structural analysis of the structure alone and is only slightly more complicated than the problem shown in Fig. 1(c).

Flexible Volume Methods

The proposed system uses a new substructuring method which is based on the observation that both the scattering and the impedance problems can be greatly simplified if more common degrees-of-freedom between the half-space and the structure are included in the interaction problem, and if the structure and the soil are partitioned in a different way. According to this partitioning, interaction occurs at all nodes of the embedded part of the structure, and the mass, damping and stiffness matrices of the structure are reduced by the corresponding properties of the excavated mass of soil. Since the general substructure theorem, Gutierrez and Chopra (1977, 1978), remains valid for this partitioning the substructure interaction analysis can now be performed as indicated by the last column of Fig. 2.

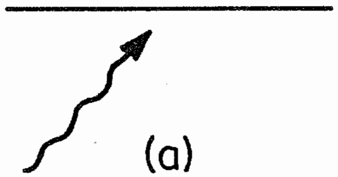
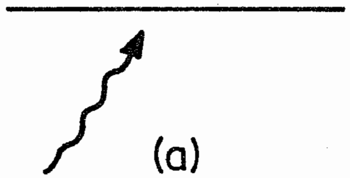
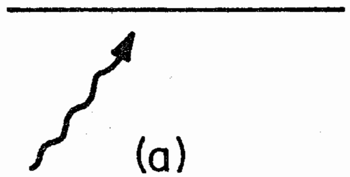
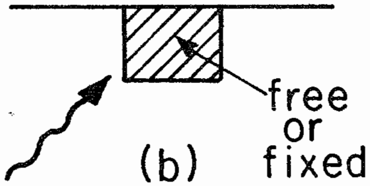
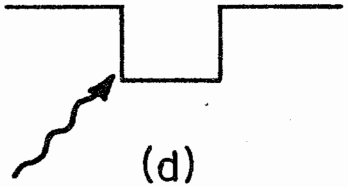
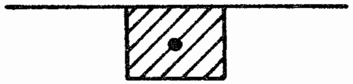
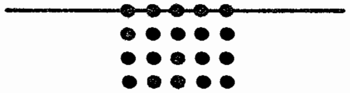
Method	Rigid Boundary	Flexible Boundary	Flexible Volume
Site Response Problem	 (a)	 (a)	 (a)
Scattering Problem	 (b)	 (d)	none
Impedance Problem • loaded node	 (c)	 (e)	 (f)
Structural Analysis	Standard	Standard +	Standard +

Fig. 2 SUMMARY OF SUBSTRUCTURE METHODS

For horizontally layered sites the site response problem can be solved by the methods described in Chapter 3. Current state-of-the-art would allow the consideration of any superposition in three dimensions of plane vertical and inclined body waves and surface waves in horizontally layered visco-elastic systems with nonlinearities considered by the equivalent linear method.

Using this approach the scattering problem is eliminated and the impedance problem is reduced to the larger but simpler problem of determining the impedance matrix for a limited number of nodes in a horizontally layered visco-elastic system. The problem is simpler than the other impedance problems shown in Fig. 2 because of the more regular boundary at the surface. As discussed below, the impedance matrix is most conveniently determined as the inverse of the dynamic flexibility matrix in the frequency domain.

The structural analysis step is only slightly complicated by the addition of more interacting nodes and follows the same procedure as for the flexible boundary methods.

Overview of Substructure Methods

Consider now the computational task involved in determining the response of a structure to say Rayleigh waves in a layered subgrade by each of the above three methods. For all methods the site response problem shown in Fig. 2(a) would have to be solved first. This effort is therefore common to all methods. In any case it is relatively simple and can be solved by the procedures described in the next chapter. The structural analysis is similar for all methods and involves essentially the same effort for all methods. Both the rigid boundary and the flexible boundary methods involve two large three-dimensional finite element analyses to solve the scattering and impedance problems (Figs. 2(b) and 2(c) or Figs. 2(d) and 2(e)). However, the flexible volume method involves only one finite element analysis (Fig. 2(f)) and this problem can, as will be shown below, be solved extremely efficiently by superposition of axisymmetric solutions. It would therefore appear that the rigid and flexible boundary methods are not competitive with the flexible volume method used in the proposed system of interaction analysis.

External Loads

The analyses of the response of structures to external loads such as impact loads acting directly on the structure do not require the solution of the site response and scattering problems since the free field is at rest with this load condition. However, the impedance and structural analysis problem must be solved. The proposed substructuring method is also convenient for this type of loading since the impedance problem is greatly simplified. Furthermore, with this substructuring method it becomes possible to consider seismic and external loads simultaneously.

Near and Far Field Motions

Up to this point it has been tacitly assumed that the interaction points in Fig. 2(f) are common to the structure and the soil. Actually, only some of them need to be common points. This is important for the applications of the flexible volume method. For example, by placing a node at some distance from the structure the motions at the location of the extra node can be determined. Also, as will be discussed in connection with one of the case studies in Section 4.3 the introduction of extra nodes near the basement makes it possible to consider secondary nonlinear effects in a region near the structure.

3. THE THEORY

The proposed system uses the flexible volume method described above and has the capabilities and limitations listed in Chapter 1. Effective methods have been developed for solving the site response problem, the impedance problem and the structural analysis problem.

3.1 General Theory

The entire analysis is performed in the frequency domain using the complex response method and finite element techniques as explained in Appendix A of Ref. 2. Thus damping is accounted for by forming all stiffness matrices from complex moduli. Transient motions are handled by Fast Fourier Transform techniques or (optionally) by random vibration techniques as for the PLUSH program, Ref. 3.

In the flexible volume substructuring method the structure(s) and the foundation are partitioned as shown in Fig. 3. In this partitioning the structure, Fig. 3(c), consists of the superstructure plus the basement minus the excavated soil and the foundation consists of the original site, Fig. 3(b), i.e., the soil to be excavated is retained with the foundation. Interaction between the structure and the foundation occurs at all basement nodes.

The equation of motion for the complete problem, Fig. 3(a) is:

$$[M]\{\ddot{\hat{u}}\} + [K]\{\hat{u}\} = \{\hat{Q}_b\} \qquad (1)$$

where [M] and [K] are the total mass and stiffness matrices, respectively. $\{\hat{u}\}$ is the vector of nodal point displacements and $\{\hat{Q}_b\}$ are the external forces. Since the source of excitement is outside the model, $\{\hat{Q}_b\}$ has non-zero elements only at the degrees of freedom corresponding to boundary nodes, which are assumed to be far away from the structure.

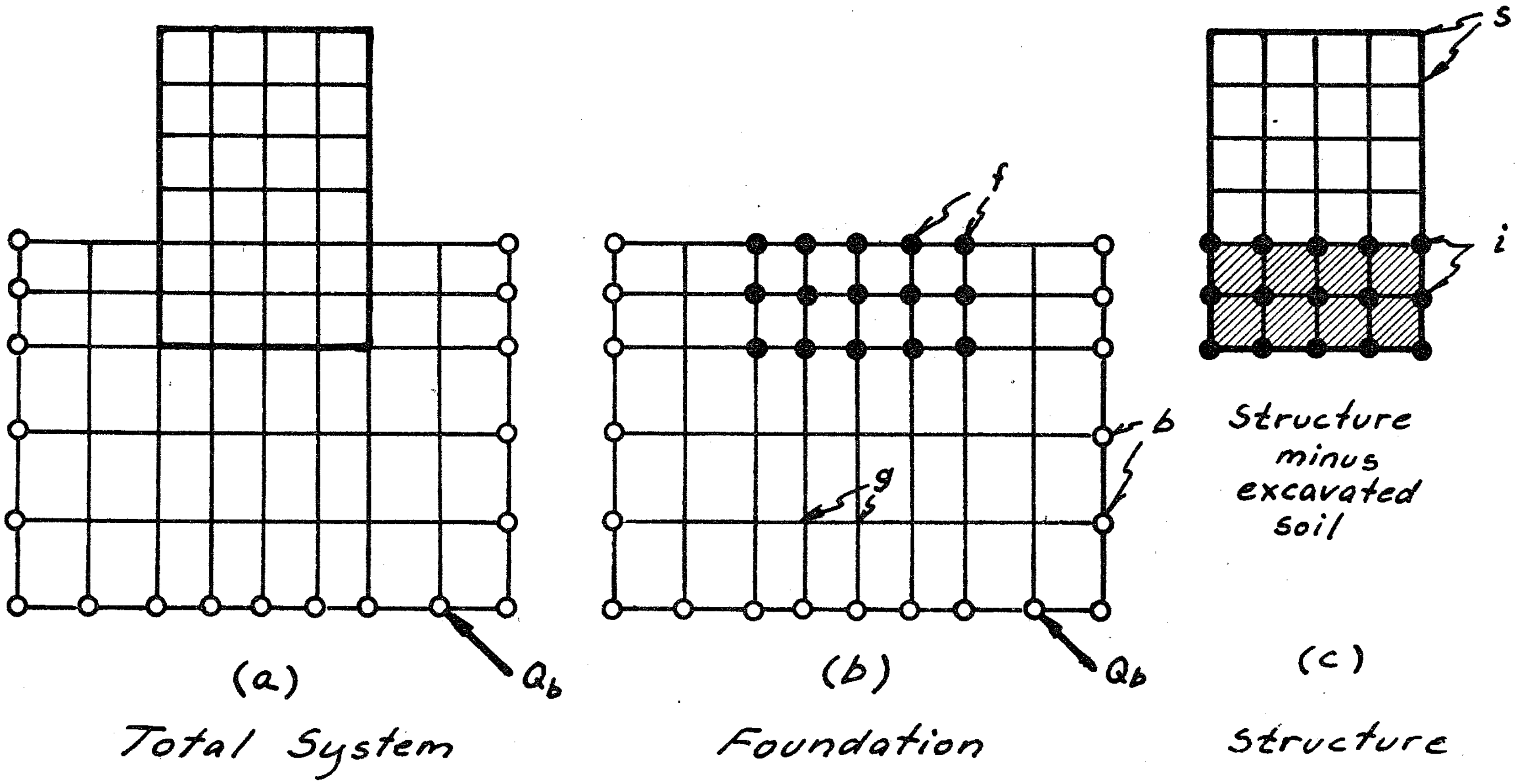


FIG. 3 SUBSTRUCTURING OF INTERACTION MODEL

For harmonic excitation of the frequency ω the load and displacement vectors can be written:

$$\{\hat{Q}_b\} = \{Q_b\}e^{i\omega t} \tag{2}$$

and

$$\{\hat{u}\} = \{u\}e^{i\omega t} \tag{3}$$

where $\{Q_b\}$ and $\{u\}$ now contains complex force and displacement amplitudes. Hence, for each frequency the equation of motion takes the form

$$[C]\{u\} = \{Q_b\} \tag{4}$$

where [C] is a complex, frequency dependent stiffness matrix:

$$[C] = [K] - \omega^2[M] \tag{5}$$

Similar equations of motion can be written for each of the substructures in Fig. 3. Introducing the following subscripts which refer to degrees of freedom (DOF) associated with different nodes:

Subscript	Nodes	
s	superstructure	
i	basement	See also Fig. 3
f	excavated soil	
b	external boundary	
g	remaining soil	

the equation of motion for the foundation (Substructure 1) in Fig. 3(b) can be written

$$\begin{bmatrix} C_{ff} & C_{fg} & C_{fb} \\ C_{gf} & C_{gg} & C_{gb} \\ C_{bf} & C_{bg} & C_{bb} \end{bmatrix} \begin{Bmatrix} u_f \\ u_g \\ u_b \end{Bmatrix} = \begin{Bmatrix} Q_f \\ 0 \\ Q_b \end{Bmatrix} \tag{6}$$

where $\{Q_f\}$ are the interaction forces from the structure in Fig. 3(c). Similarly, the equation for the structure (Substructure 2) in Fig. 3(c) can be written:

$$\begin{bmatrix} C_{ss} & C_{si} \\ C_{is} & (C_{ii} - C_{ff}) \end{bmatrix} \begin{Bmatrix} u_s \\ u_f \end{Bmatrix} = \begin{Bmatrix} 0 \\ -Q_f \end{Bmatrix} \tag{7}$$

where compatibility of displacements ($u_i=u_f$) and equilibrium ($Q_i+Q_f=0$) has been enforced. The term ($C_{ii}-C_{ff}$) simply indicates the stated partitioning according to which the stiffness and mass of the excavated soil is subtracted from the stiffness of the structure.

Assuming that the external boundary is very far away from the structure the equation of motion for the free field (Substructure 1) can be written:

$$\begin{bmatrix} C_{ff} & C_{fg} & C_{fb} \\ C_{gf} & C_{gg} & C_{gb} \\ C_{bf} & C_{bg} & C_{bb} \end{bmatrix} \begin{Bmatrix} u'_f \\ u'_g \\ u'_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ Q_b \end{Bmatrix} \tag{8}$$

where $\{u'\}$ are the free-field motions (the solution to the site response problem). Subtraction of Eq. 8 from Eq. 6 yields:

$$\left[\begin{array}{cc|c} C_{ff} & C_{fg} & C_{fb} \\ \hline C_{gf} & C_{gg} & C_{gb} \\ C_{bf} & C_{bg} & C_{bb} \end{array}\right] \left\{\begin{array}{c} r_f \\ \hline r_g \\ r_b \end{array}\right\} = \left\{\begin{array}{c} Q_f \\ \hline 0 \\ 0 \end{array}\right\} \tag{9}$$

where $\{r\} = \{u\} - \{u'\}$ are the interaction displacements. By partitioning Eq. 9 as indicated $\{r_g\}$ and $\{r_b\}$ can be eliminated and $\{Q_f\}$ can be expressed in the form:

$$\{Q_f\} = [X_f]\{r_f\} = [X_f](\{u_f\} - \{u'_f\}) \tag{10}$$

The frequency dependent matrix $[X_f]$ is called the impedance matrix and an effective method for determining this matrix without using the large matrix in Eq. 9 will be described in Section 3.3.

Substitution of Eq. 10 into Eq. 7 yields:

$$\begin{bmatrix} C_{ss} & C_{si} \\ C_{is} & (C_{ii} - C_{ff} + X_f) \end{bmatrix} \begin{Bmatrix} u_s \\ u_f \end{Bmatrix} = \begin{Bmatrix} 0 \\ [X_f]\{u'_f\} \end{Bmatrix} \quad (11)$$

from which the final motions of the structure can be determined.

Thus the solution of the soil-structure interaction problem has been reduced to three steps:

1. Solve the site response problem

   to determine the free field motions $\{u'_f\}$ within the embedded part of the structure, see Fig. 2(a).

2. Solve the impedance problem

   to determine the matrix $[X_f]$, see Fig. 2(f).

3. Solve the structural problem

   This involves forming the complex stiffness matrices and load vector shown in Eq. 11 and solving this equation for the final displacements.

3.2 Site Response Analysis

As previously stated the original site is assumed to consist of horizontal soil layers overlying a uniform half-space. All material properties are assumed to be visco-elastic. However, the stiffness and damping of each layer are adjusted by the equivalent linear method, see Ref. 2, p. A-7 to p. A-8.

As described in Chapter 4 of Ref. 2 methods have been developed to solve the site response problem corresponding to inclined body waves and surface waves.

With the proposed system only the displacement of the layer interfaces where the structure is connected are of interest. Thus, and this is possible for all of the above wave types, displacement amplitudes will be expressed on the form

$$\{u_f'(x)\} = \{U_f\}e^{i(\omega t - kx)} \qquad (12)$$

where $\{U_f\}$ is a vector (mode shape) which contains the interface amplitudes at the control point (x=0) and k is a complex wave number which expresses how fast the wave propagates and decays in the horizontal x-direction. Effective discrete methods have been developed at the University of California, Chen (1980), for determining appropriate mode shapes and wave numbers corresponding to control motions at any layer interface for inclined P-, SV- and SH-waves, Rayleigh waves, see Ref. 2, and Love waves. Any combination of such waves can be applied simultaneously.

3.3 Impedance Analysis

The impedance matrix $[X_f]$ is similar to a dynamic stiffness matrix for the interaction nodes. Thus it can be determined as the inverse of the dynamic flexibility matrix, [F], for these nodes. The columns of [F] can be determined by successively applying unit amplitude loads of each DOF and determining the displacement amplitudes of the nodes.

Since the foundation is a simple layered system these displacements can be determined from the axisymmetric finite element model shown in Fig. 4. This model consists of a single column of special cylindrical elements which has three degrees of freedom at the center nodes and transmitting boundaries, Kausel (1975), at the perimeter. The radius of the column is chosen smaller than the smallest distance between nodes in the basement and the unit load can be applied vertically or horizontally at any of the central nodes. From this model displacement amplitudes can be obtained both at the central nodes and at any point outside the cylindrical elements, i.e., at all interaction (basement) nodes.

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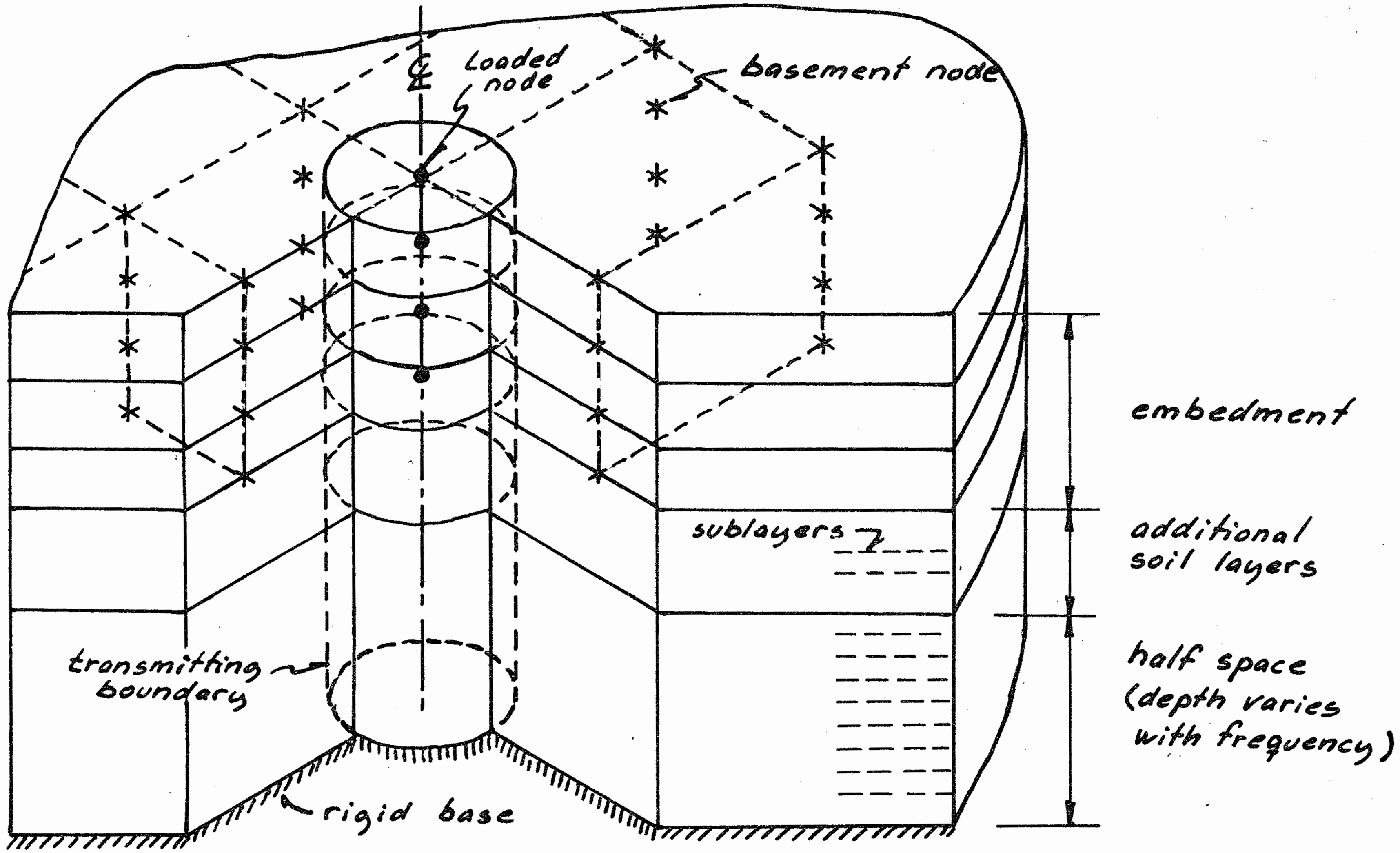


FIG. 4 AXISYMMETRIC MODEL FOR IMPEDANCE ANALYSIS

The number of solutions required to construct the entire flexibility matrix is surprisingly small. Thus for a basement containing say 125 nodes but connected only at five layer interfaces as shown in Fig. 6 only ten solutions (loads at two DOFs at each layer interface) of the axisymmetric problem in Fig. 4 are required to construct the complete flexibility matrix.

The inversion of the flexibility matrix to form the impedance matrix $[X_f]$ is a formidable numerical problem. However, program SASSI employs a special method for obtaining this matrix without a complete inversion procedure.

In the latest version of the program the rigid boundary of the model shown in Fig. 4 has been replaced by a transmitting boundary which effectively prevents reflection. Thus a semi-infinite half-space is simulated with high accuracy.

Modification for External Loads

The equation of motion, Eq. 11, considers only seismic forces. External loads at the superstructure and basement nodes can be considered by simply adding the amplitudes of these forces to the load vector (right hand side of Eq. 11) at each frequency.

3.4 Structural Analysis

With the exception of forming the load vector and modifying the complex stiffness matrix for the structure by subtracting the corresponding matrix for the excavated soil and adding the impedance matrix as shown in Eq. 11, the structural analysis is standard and the computational effort is essentially the same as for a complex response analysis of the unmodified structure on a rigid foundation.

4. THE IMPLEMENTATION

A modular computer code, SASSI, is currently being developed to perform the operations described in Chapter 3. The code is arranged specifically for practical applications and has the following characteristics:

a) The site response analysis, the impedance analysis and the formation of the basic stiffness and mass matrices for the structure can be performed separately and the results stored on magnetic tapes or disks.

b) Thus, if the seismic environment, the external loads, the soil properties or the arrangement of the superstructure are changed only part of the computations have to be repeated.

c) The final solution is stored (in the form of transfer functions) on a magnetic tape from which specific information can be extracted when required without recomputation of the entire solution.

d) Both deterministic (time histories) and probabilistic results can be obtained from the above tape.

4.1 Layout

The general layout of the system is shown in Fig. 5.

HOUSE

The basic frequency independent stiffness matrices [M] and [K] for the structure and the excavated soil are formed by program HOUSE and stored on Tape 1.

SITE

The site response problem is solved by program SITE which from the site properties and the nature of the control motion produces the mode shapes and wave numbers in Eq. 12. This program also provides information

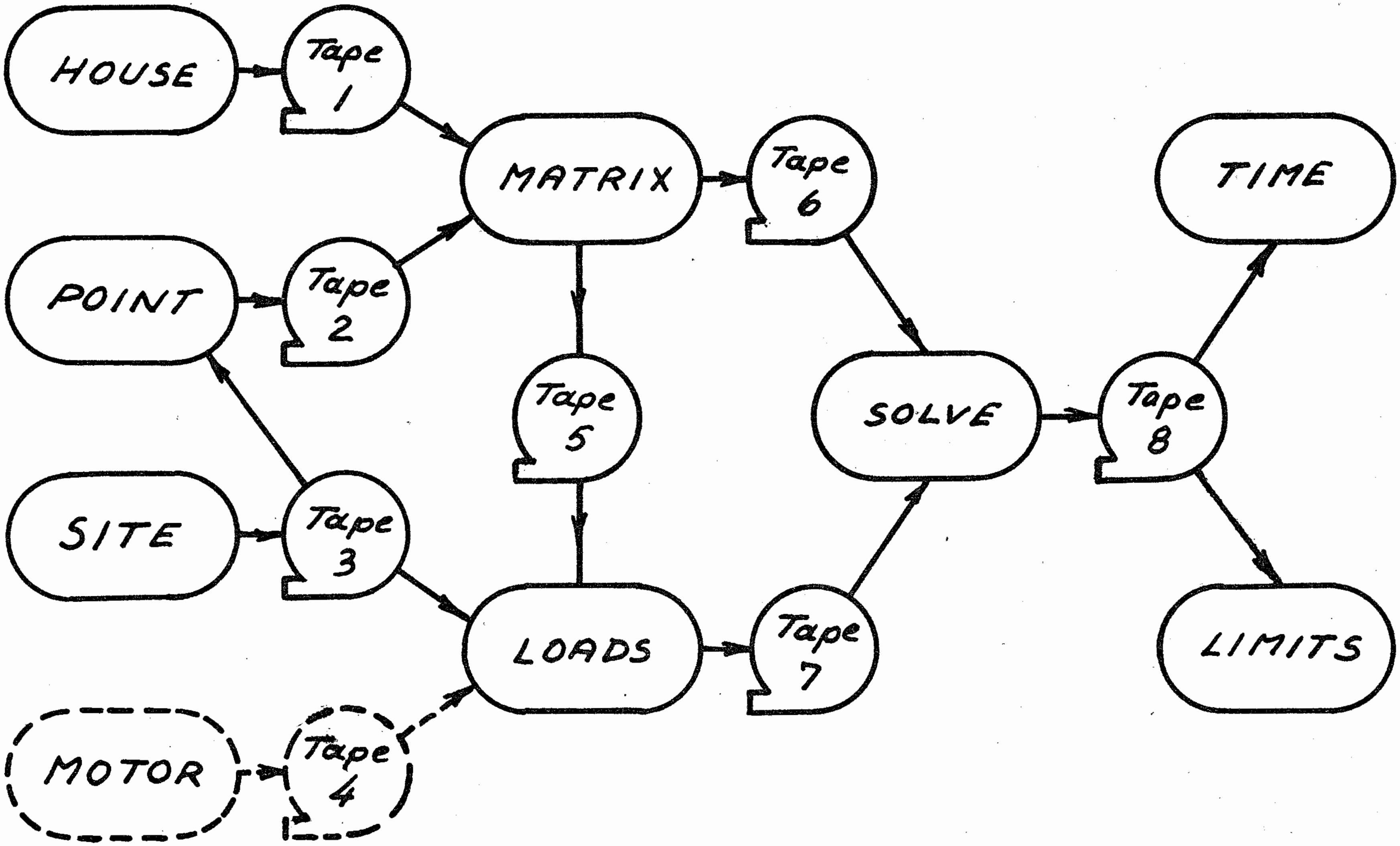


Fig. 5 - Layout of System SASSI

required to compute the transmitting boundaries used in solving the impedance problem. The results are written on Tape 3. The actual time history of the control motion is not required at this time; only the composition of the wave types causing the motion.

POINT

The flexibility matrix, [F], for the interaction nodes is formed by program POINT for each frequency of interest and the matrices are stored on Tape 2. This program requires as input part of the information on Tape 3. Thus program SITE must be executed before program POINT.

MOTOR

This program forms elements of the load vector in Eq. 11 which corresponds to external forces such as impact forces acting on the structure or forces from rotating machinery within the structure. The generated information is stored on Tape 4.

MATRIX

Using input from Tapes 1 and 2 the program MATRIX forms for each frequency the impedance matrices [X(ω)]. These are written on Tape 5. It also forms the modified complex stiffness matrices in Eq. 11, triangularizes them and stores the triangularized forms on Tape 6.

LOADS

This program computes the load vector in Eq. 11 for each frequency and stores them on Tape 7.

SOLVE

Program SOLVE reads the reduced stiffness matrices from Tape 6 and the load vectors from Tape 7. It then performs the backsubstitution to obtain the total displacement amplitudes from Eq. 11. These amplitudes are actually transfer functions from the control motion to the final motions. The results are stored on Tape 8 which now contains the complete solution in terms of transfer functions.

For a typical problem the above operations need to be performed at only 20 to 40 frequencies. The remaining values of the transfer functions are determined by interpolation in the frequency domain using a new effective interpolation procedure which has been developed as part of the current research program. The actual interpolations are performed in the post processors TIME or LIMITS.

TIME

The main purpose of this (deterministic) post-processor is to produce time histories of selected output variables such as accelerations, bending moments, etc. It may also output transfer functions and response spectra.

In its basic mode the program reads the acceleration time history of the control motion from cards and transforms it to the frequency domain using Fast Fourier Transform techniques. It then reads the uninterpolated transfer functions from Tape 8 for selected output motions, performs the interpolation and the convolution with the control motion and returns to the time domain using the inverse Fast Fourier Transform algorithm. Details of the technique are provided in Appendix A of Ref. 2. The resulting time histories of acceleration may be output directly or converted to output response spectra.

Many other output functions are planned for program TIME. However, these will be determined as needs develop.

LIMITS

This (probabilistic) post-processor is in many respects similar to program TIME. However, instead of accepting the time history of the control motion it accepts a power or response spectrum of this motion. It then evaluates the probabilistic response of the structure according to the techniques used in program PLUSH, see Refs. 3 and Romo et. al. (1977).

The output consists of mean values and confidence limits on selected design parameters such as maximum accelerations and bending moments. The program can also produce mean values and confidence limits of acceleration response spectra.

Experience with program PLUSH has shown that probabilistic output through program LIMITS can be produced at a small fraction of the cost required to produce deterministic output through program TIME. Furthermore, the probabilistic procedure eliminates the need to develop time histories of motion which fit given design response spectra.

4.2 Operational Features

The system shown in Fig. 5 has been specifically designed to provide maximum flexibility and economy for practical applications.

Clearly, on any given project most of the programs have to be executed. However, if changes occur in the design parameters only parts of the system have to be re-executed. Consider the following examples:

Change in Control Motion

Suppose results are required for a different time history (or response spectrum) of the control motion. Then, as long as the nature of the seismic environment (i.e., the type of wave field) is not changed, only program TIME (or LIMITS) has to be re-executed.

Change in Seismic Environment

Suppose the structure was originally analyzed for the effects of vertically propagating body waves and that results are required for the case of incident Rayleigh waves causing the same motion at the control point on the free field. In this case only part of program SITE and all of programs LOAD, SOLVE and TIME (or LIMITS) have to be re-executed. This is so because the information on Tapes 2, 5 and 6 remains unchanged.

Change in Superstructure

If changes are made in the superstructure only HOUSE, part of MATRIX, SOLVE and TIME (or LIMITS) have to be re-executed.

Post-Processors

Since the data on Tape 8 is independent of the manner in which the frequency content of the control motion is specified, the user has at any time the choice of deterministic and probabilistic analysis.

Also, since the post-processors can be restarted from Tape 8, the user has at any time the option of outputting only the data which he actually needs. This is especially important for three-dimensional analysis for which the printing cost and volume of the complete solution easily become unmanageable.

Post-processors which can perform these tasks are currently in different stages of development. Nearly completed is a post-processor which computes time histories of stresses and strains in solid elements and internal forces in beam and plate elements. Procedures have also been developed for computing stresses and displacements in the ground outside the excavated volume.

Computer Costs

Three-dimensional finite element analysis is always costly. Hence, the system will not be exactly cheap to run. However, since the proposed system does not require that a large soil mass be modeled by finite elements, true three-dimensional SSI analysis will be within reach.

It is currently estimated that the SSI analysis of a typical embedded structure will cost less than three times the cost of analyzing the structure and the basement alone. The cost depends strongly on the number of connected nodes in the basement (dots in Fig. 5(f)). Thus the cost of analyzing very deep foundations, say large pile groups, may be prohibitive, unless advantage can be taken of symmetry.

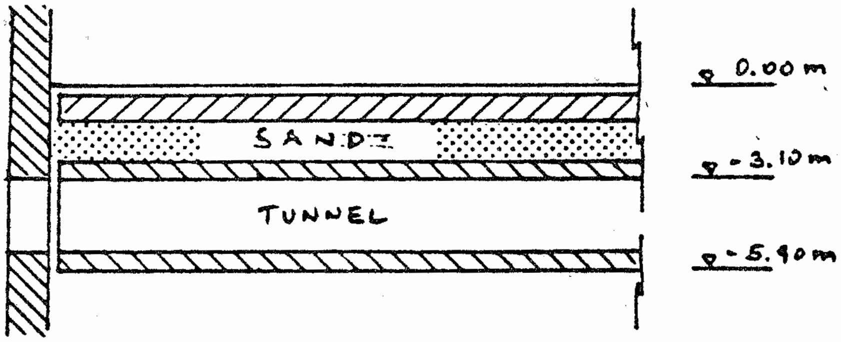
Research is currently in progress to improve the efficiency of the system, especially in the area of computing the impedance matrix $\{X_f\}$ which is the most expensive part of the analysis (up to 85% of the total cost). Recently an efficient in-place inversion subroutine was developed for the computation of the impedance matrix from the flexibility matrix. This routine which takes full advantage of the symmetry of these matrices reduces the cost of the inversion. Currently methods of taking advantage of symmetry (in loads, in superstructure or in basement only) are being implemented.

4.3 Case Studies

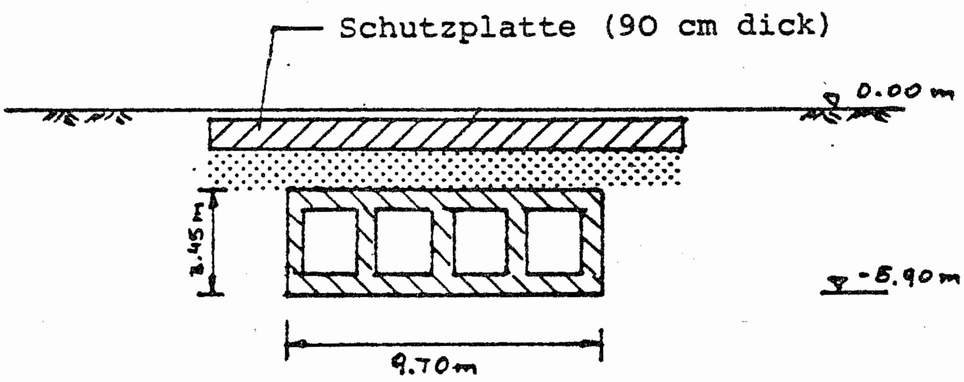
Several case studies have been performed using the SASSI approach.

Airplane Impact Problem

The cable tunnel shown in Fig. 6 was analyzed for the effect of airplane impact on the protective slab shown above the tunnel. The soil profile and finite element model used are shown in Figs. 7 and 8. Advantage was taken of symmetry about two vertical planes and, as shown in Fig. 8, the load was assumed to act vertically at the center of the slab with the time history shown in Fig. 9. Computed maximum vertical displacements are shown in Fig. 10. The computed time history of vertical velocity of Node 4 (point of load application) is shown in Fig. 11. The results show that the protective slab is very effective in reducing the motions of the tunnel.



LÄNGSSCHNITT



QUERSCHNITT

Fig. 6 Cable Tunnel

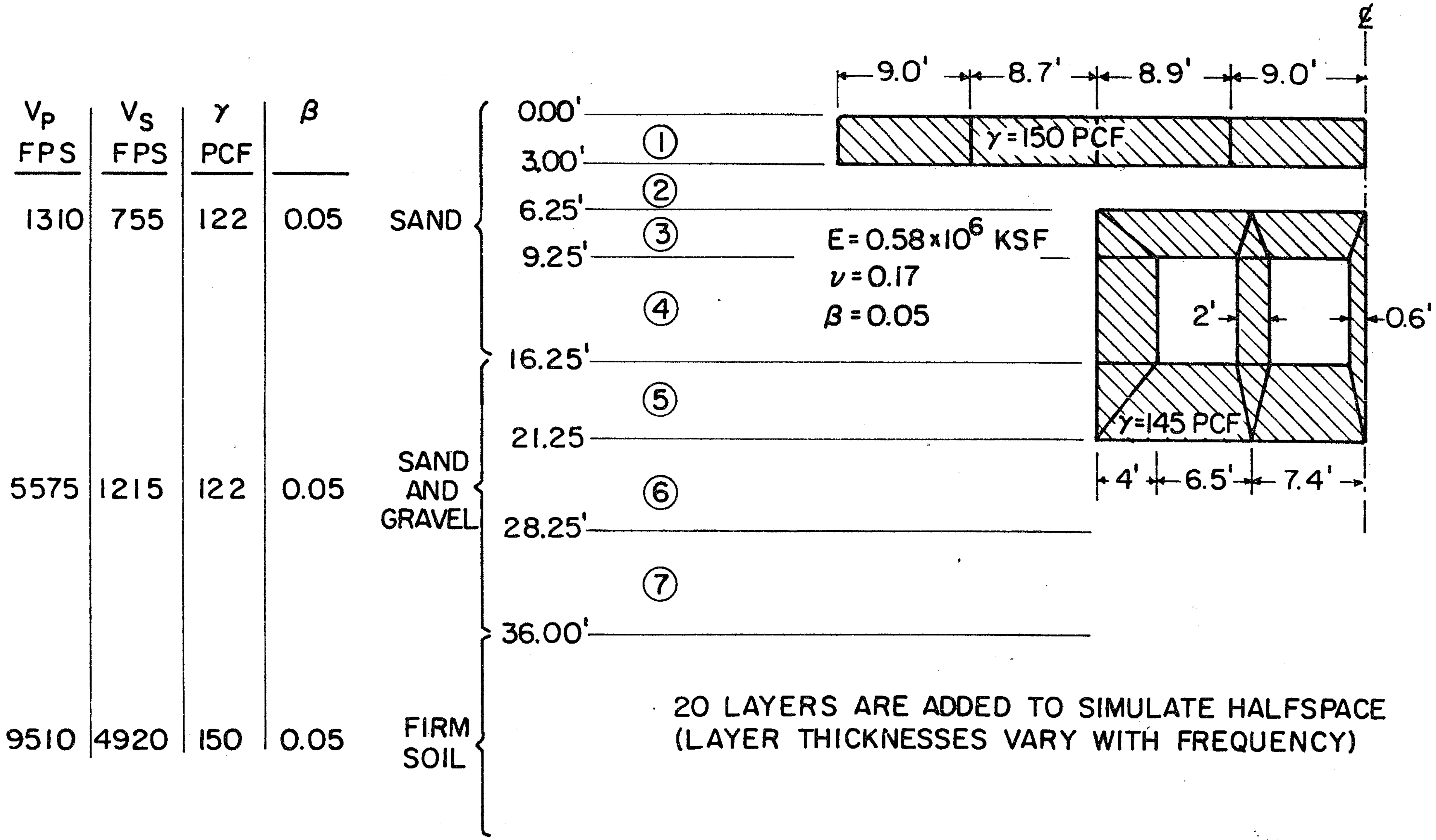


Fig. 7 Soil Profile and Model Cross Section

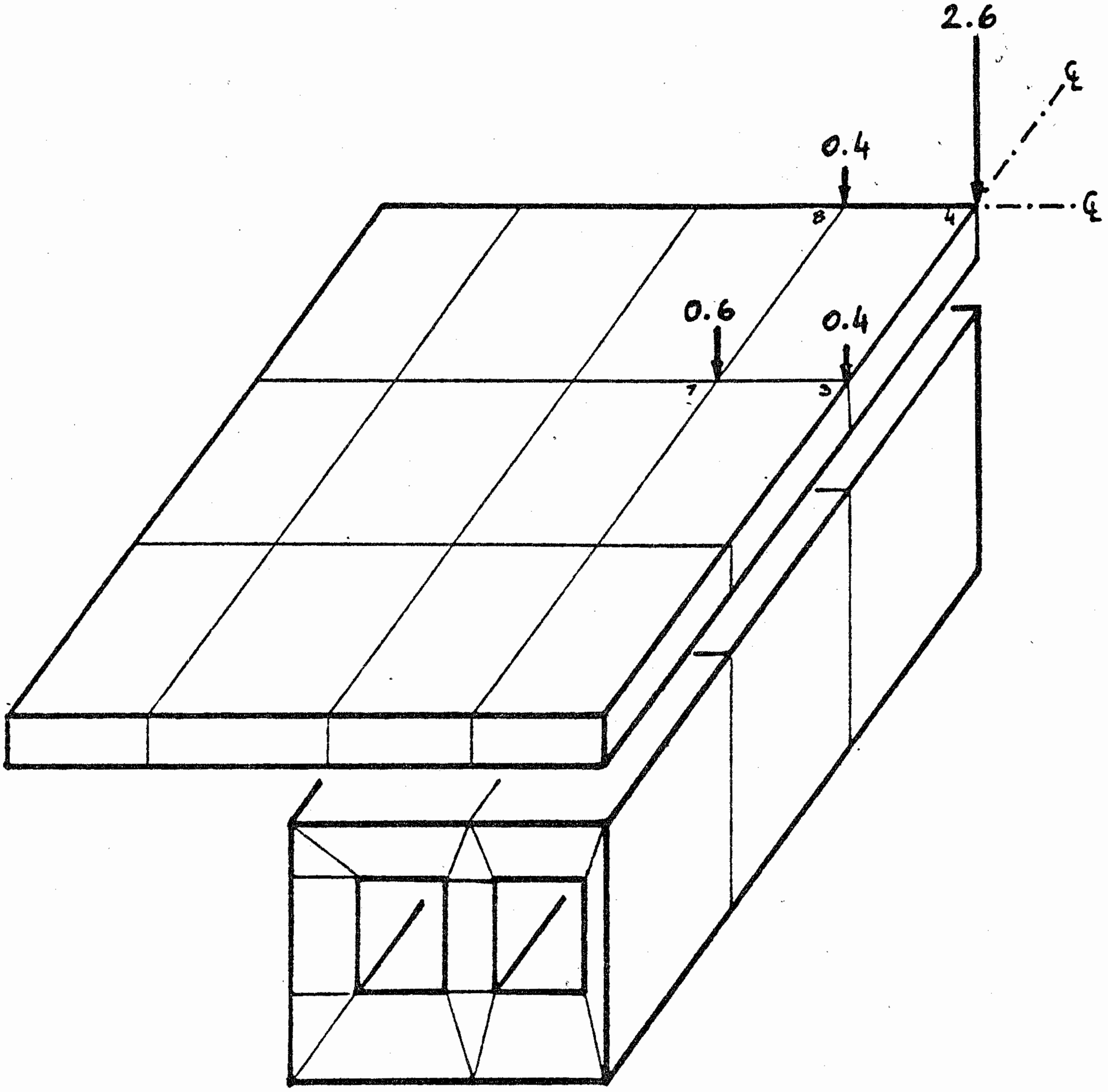


Fig. 8 Finite Element Model and Load Factors