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SOIL STRUCTURE INTERACTION CALCULATIONS: A COMPARISON OF METHODS

Lawrence Wight
Maurice Zaslowsky

July 22, 1976

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LAWRENCE LIVERMORE LABORATORY
University of California Livermore, California 94550

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SOIL STRUCTURE INTERACTION CALCULATIONS: A COMPARISON OF METHODS

Abstract

Two approaches for calculating soil structure interaction (SSI) are compared: finite element and lumped mass. Results indicate that the calculations with the lumped mass method are generally conservative compared to those obtained by the finite element method. They also suggest that a closer agreement between the two sets of calculations

is possible, depending on the use of frequency-dependent soil springs and dashpots in the lumped mass calculations. There is a total lack of suitable guidelines for implementing the lumped mass method of calculating SSI, which leads to the conclusion that the finite element method is generally superior for calculative purposes.

Introduction

Nuclear power plants in the United States are currently licensed to operate after the Nuclear Regulatory Commission (NRC) has reviewed their design in detail. The NRC has many regulations that establish licensing policy, and it continually attempts to improve this policy by modifying regulations to account for an improved understanding or for new design methods. One excellent example is the NRC policy on seismic design. Here, the improved policy requires the accumulation of data from large, infrequent earth-

quakes. One particularly important aspect of seismic design is the effect, on the soil, of the motion of a massive, stiff structure. Called soil structure interaction (SSI), the effect can result in different soil motions compared to the free field; thus, it can significantly affect structural response.

Although the NRC does not now have an official policy on SSI, it describes its method of license review in section 3.7.2 of the Standard Review Plan (SRP).¹ Among the topics discussed in this section, is a

Table 1. Acceptable methods for SSI calculations.

Method	Rock [†] foundation	Deep embedment case ^{††}	Soil foundation [*] Shallow embedment case		
			Deep soil foundation with uniform properties	Deep soil foundation with layered properties	Shallow soil foundation
Single lumped mass spring	x		x		
Multiple lumped mass spring ^{**}	x		x	x	x
Finite element ^{**}	x	x	x	x	x

Table 2. SSI calculative methods selected for comparison.

Method	Deep embedment case ^{††}	Soil foundation [*] Shallow embedment case	
		Deep soil foundation with layered properties	Shallow soil foundation
Single lumped mass spring	x	x	x
Finite element ^{**}	x	x	x

* Soil foundation means the depth of soil between the bottom of the foundation slab and the base rock.

[†]A medium for which the soil structure interaction effect is negligible or, alternatively, a medium with a shear-wave velocity ≥ 3500 fps.

^{**}Or equivalent.

^{††}Deep embedment: actual embedded depth $> 15\%$ of the least base width or other appropriate value to be justified.

summary of acceptable methods to calcu- matrix in Table 1 reflects the rela-
late SSI (Table 1). Note that the tive advantages and disadvantages

among available methods, depending on site homogeneity and embedment depth. To establish the technical basis for a regulatory guide on this subject, the NRC Office of Standards Development (OSD), Engineering Standard Branch, requested Lawrence Livermore Laboratory (LLL) to evaluate several SSI calculative methods. This report describes the results of our evaluation.

In discussions with OSD, we established that only the most controversial elements in the matrix, or (as indicated in Table 2) those elements in which the use of the single lumped mass spring was not permitted, would be assessed. Also, since the multiple, lumped mass approach of calculating SSI is so infrequently used in licence applications, the evaluation was limited to a comparison of the single, lumped mass approach and

finite element approach. We further agreed that a good technical basis could be established by a parametric survey, covering sites of different stiffnesses, thicknesses, and depths of structural embedment. The results would then clearly indicate the consequences of the lumped mass vs finite element choice of calculative method.

In the next section of the report, we discuss the nature and consequences of SSI. Then we describe various lumped mass and finite element methods, after which we outline our approach to the comparison. Finally, we discuss the results in detail and state our conclusions. Most of the data on which we have based our comparisons are shown in Appendices A, B, and C. An SI conversion table is given in Appendix D for the benefit of readers who may wish to convert the non-SI units in the report to the metric system.

Phenomenology of Soil Structure Interaction

Soil structure interaction has been recognized as an important design consideration for almost 100 years.² Only recently has it gained wide attention. This is largely because the integrity of certain structures, such as nuclear power plants, has become very important. The SSI is a significant consideration, because it results in a modification of the seismic input to a structure: there-

fore, its understanding is crucial to design.

The SSI is a function of three factors: (1) modal characteristics of the structure, (2) fixed-base structural frequencies relative to the excitation frequencies, and (3) soil properties. The last factor is particularly germane, because the large soil strains developed around a structure during excitation soften the

soil moduli. This has an appreciable effect on the structural response. In general, the interaction affects a structure by modifying the foundation or basemat excitation (compared to free field) and by lowering the apparent fundamental frequency of the structure. The interaction is especially significant for embedded structures, where the increased contact area causes a further modification of the structural frequencies and increases the radiated energy.

There are numerous examples of how soil structure interaction has modified the structural response. In this section, we attempt to review some of the experimental evidence to indicate the significance of the topic.

Scavuzzo and Raftopoulos³ examined the effects of the San Fernando (1971) earthquake on the Hollywood storage building. Response measurements, made at the rooftop and basement, were compared with both free field and analytical results. The authors concluded that the interaction effects alter the free field motion and initiate rocking. Preliminary results of another comparison, made at the San Onofre Nuclear Power Plant, indicated an amplification of ground motion at the basemat, caused by soil structure interaction.

Jennings and Kuroiwa⁴ performed SSI experiments on a building at the

California Institute of Technology campus. They used vibration generators on the top story. Their results indicated that this method, when coupled to measuring foundation motion and nearby soil surface movements, could be a means of studying SSI.

Tajimi et al.⁵ measured the response of the containment building of a partially embedded reactor during five earthquakes. In this case, the measurements were used to select input for the SSI model.

Hradilek et al.⁶ reported on the basemat response to an underground nuclear detonation in Nevada. They also reported on a tower's response, in Mexico City, to the 1962 earthquake. In both cases, their measurements showed that SSI was present and was a strong function of the structural and excitation frequencies. Seed⁷ reported on the effects, at the Humboldt Bay reactor, of the Ferndale earthquake. The data, which included free field and structural response, were used to check his SSI calculations. The agreement was excellent.

Muto and Omatsuzawa⁸ used data from the 1970 Japanese earthquakes, which affected a BWR-type reactor, to compare the experimental and calculated responses. The recorded time history was used as input. And the responses, as measured and calculated, compared well. The authors used complex analysis and frequency-dependent

compliances in their model. Their results indicate the importance of soil structure interaction.

These numerous examples are available to indicate the importance and effect of SSI. Sufficient experimental data do not exist to cover all possible cases, a fact that has severely limited the develop-

ment of credible theories. Because of this shortage of good strong-motion data, it is presently impossible to quantitatively determine the adequacy of an SSI calculation method. Conclusions can only be based on comparative calculations. This is a major limitation in the present state of the art.

Lumped Mass-Spring Method

This SSI calculative method models the interaction effects with soil springs and dashpots, through which soil motions are transmitted to the structure foundation. The spring and dashpot parameters are derived from the analytical solution of the response of a rigid circular plate, or an infinite strip bonded to the surface of an elastic half space, to harmonic excitation. Although these parameters depend on excitation frequency, there are various ways to determine average parameters that are frequency independent. The use of average frequency-independent springs and dashpots is one, and because of their simplicity the springs and dashpots are commonly used for design purposes.

COMMON APPROACHES

In this section, we review and compare several of the most common

methods for determining parameters with frequency-independent springs and dashpots. We also describe the methods of calculating structural response, and we discuss the requirements in relation to soil structure interaction. Both topics are important, since they determine the credibility of the interaction calculation. At the end of the section, we discuss the judgment required of an analyst to implement these approaches.

Whitman Soil Springs

In 1967, Whitman (Ref. 9) published the first of several papers describing his method of determining the parameters of soil springs and dashpots. These papers have received such wide circulation and attention that the Whitman approach is often identified as the soil spring method. His contributions include new suggestions

Table 3. Equivalent lumped parameters for analyzing circular foundations

	Vertical	Swaying	Rocking	Twisting
Spring constant	$\frac{4GR}{1-\mu}$	$\frac{8GR}{2-\mu}$	$\frac{8GR^3}{3(1-\mu)}$	$\frac{16GR^3}{3}$
Mass ratio B	$\frac{M(1-\mu)}{4\rho R^3}$	$\frac{M(2-\mu)}{8\rho R^3}$	$\frac{3I(1-\mu)}{8\rho R^5}$	$\frac{I_t}{\rho R^5}$
Damping ratio	$\frac{0.425}{\sqrt{B}}$	$\frac{0.29}{\sqrt{B}}$	$\frac{0.15}{(1+B)\sqrt{B}}$	$\frac{0.50}{1+2B}$
Effective mass	$\frac{0.27M}{B}$	$\frac{0.095M}{B}$	$\frac{0.24I}{B}$	$\frac{0.24I_t}{B}$

M = mass of foundation; I = moment of inertia about horizontal axis; I_t = moment of inertia about vertical axis; G = shear modulus of soil; ρ = soil density; R = foundation radius; μ = Poisson's ratio of soil.

for the damping factor and embedment effects.¹⁰

The damping factor suggestions are of particular note, since they are based on experimental data; however, the recommended damping factors represent a lower bound in the data and are therefore conservative. Whitman's equations, as applied to circular foundations, are summarized in Table 3. The embedment equations are solely analytical, since there are little data available in this area. Whitman recommends a set of results in which embedment is based on an approximate theory that adds a stiffness function of a plate to the half-space stiffness function.

Newmark Approach

Newmark & Rosenblueth,¹¹ using the results of Lysmer¹² and others, present a set of expressions for spring and dashpot constants for circular and rectangular foundations. In Newmark's formulation of the problem, the soil springs and dashpots are appropriate for low frequencies. In Whitman's, the springs and dashpots are appropriate for higher frequencies. Also, Newmark introduces the concept of a virtual mass of soil that permits a better adjustment in the resonant frequency range, i.e., the dashpot constants are smaller by the presence of the vertical mass.

The Newmark approach is valuable for certain applications. But we feel that the Whitman approach is more appropriate for the response of nuclear power plants. In such cases, we require an accurate, high frequency response that is available only through the Whitman approach. Further, the need for a virtual soil mass is questionable.¹⁰ Further, it has been our experience that the inclusion of an appropriate soil mass does not significantly change the structural response.

Hall and Kissenpfennig Approach

A method suggested by Hall and Kissenpfennig¹³ for embedded structures is to use a "hybrid" approach that utilizes aspects of both the finite element and lumped mass methods. The salient feature is to couple the horizontal and rocking modes of the foundation vibration. The method used in Ref. 13 is to replace the soil by a one-dimensional soil column, which can be represented by three springs and two masses, as shown in Fig. 1. With the springs distributed in this manner, the horizontal motion couples to the rocking motion. Masses M_1 and M_2 are sufficiently large so that their response represents free field motion. This method differs little from the one adopted by Whitman for an embedded structure and from the model used in this study.

The basis of the Hall's embedment spring constants is described in a paper by Johnson et al.¹⁴ While the approach has a great deal of merit, it is difficult to generalize from their results. First, they assumed an elastic soil model. Second, they did not examine a wide range of site conditions. Nevertheless, their method does provide a rational way to develop spring constants on a case-by-case basis, and it is a valuable supplement to the lumped mass approach.

Parmelee Approach

Parmelee (Ref. 15) recently presented another method to determine parameters with frequency-independent springs and dashpots. He first takes the arithmetic mean with respect to frequency. Then, by means of a linear regression analysis, he determines the parameters' functional dependence

Fig. 1. One lumped parameter model for embedded foundations.

on Poisson's ratio. He also presents a new approach to embedment. He, like Whitman, determines that embedment effects are a function of the embedment/width ratio, δ . However, unlike Whitman's embedment theories, Parmelee's approach to embedment is theoretical in nature and not based on data. For this reason, we favor the Whitman method, although we look forward to comparing other embedment theories with reliable SSI data.

In summary, we feel that of the frequency-independent approaches, Whitman's is presently the most credible and most widely accepted. The next section addresses the effect of calculational method on the results.

RESPONSE CALCULATION METHODS

Generally speaking, there are three methods for calculating structural response: (1) mode superposition, (2) time-history direct integration, and (3) complex response. These methods have very specific advantages and disadvantages when applied to SSI.

The unique feature of an interaction calculation is that the soil dashpots are such that the damping is nonproportional. This means that the system does not possess classical modes, because the transformation that diagonalizes the stiffness and mass matrices does not diagonalize

the damping matrix. Consequently, the usual mode superposition techniques for calculating structural response are inappropriate.

However, it has been frequently suggested that an adequate approximation of the structural response can be obtained by developing an appropriate diagonal damping matrix. Several methods for deriving this diagonal damping matrix have been proposed. They are based largely on intuitive grounds, such as linear averaging of damping, strain energy averaging of damping, or by matching the response of the coupled and uncoupled systems at certain frequencies. Despite a lack of firm basis, the diagonalization approximations are significant, because evidence indicates that the errors in the approximation are conservative,¹⁶ and because a normal mode solution is economical.

In the time-history approach, there are no difficulties with non-proportional damping, since the solution marches in time without the need to uncouple the equations of motion. However, there is significant variability in the way the damping matrix is formulated. Rayleigh damping is the most common approach; here, the damping factors are assumed to be proportional to the mass and stiffness matrices. But the difficulty is that the approach permits specified damping

in only two modes. Further, it is hard to determine the damping in the other modes, without resorting to the complex methods of Foss.¹⁷ Other improved approaches to determine the damping matrix have been proposed.¹⁸ But the best approach is to specify the damping as complex damping.^{16,19} This method is not available as an SSI calculational tool for time-history approaches. But it has the advantage of resulting in frequency-independent damping, which is considered to be the most appropriate method for soils and structures.

Further disadvantages of the time-history approach are that it requires large amounts of computer time, and in some algorithms additional numerical damping is often introduced.²⁰ This latter point is particularly significant for the high-frequency response design of nuclear power plants. The additional damping modifies the high-frequency response and thus introduces errors in the design spectrum. Nevertheless, the problem can be minimized by paying careful attention to the integration scheme.

The complex-response method is the most appropriate one for solving problems that include the frequency-dependency of soil parameters. It involves a solution in the frequency domain; here, the harmonic components of the input are passed through a filter that corresponds to the transfer

function of the system. An inverse transformation to the time domain, and finally a superposition of the harmonics, results in the system response. The three most common complex methods are:

- Transform the equations of motion to $2(N+2)$ space and, by the Foss method, obtain the $2(N+2)$ complex eigenvalue solution. The response is then a superposition of the complex model responses.
- Use the Laplace transform in lieu of the Foss method. The basic difference here is in determining the natural frequencies, damping ratios, and base acceleration.
- Apply the fast Fourier transform method to the equations of motion.

The principle disadvantage of the complex-response method lies in its restriction to linear problems. However, various iterative schemes to account for soil nonlinearities are available.

SUBJECTIVE DECISIONS

This is an important subject, because the determination of a superior SSI method could stem from ease-of-implementation considerations. The first subjective decision is how to calculate the shear modulus for a

layered site. The value of the modulus varies as a function of depth below the reactor site; also, it depends on the materials in that sublayer. Whitman addresses this point by placing bounds on the modulus. But he did not intend them to be generally appropriate — they should be checked by calculations for specific sites. One check is to calculate average moduli based on corrected, free-field soil response. But then the decision of how to average the moduli (arithmetic or weighted), and if they are weighted how deep to average them, has to be made. Also, decisions regarding the depth require an anticipation of the extent of the interaction stress bulb. This is particularly difficult for layered sites.

A second decision concerns the value of the damping coefficient. As we have shown, a variety of approaches are available to establish damping coefficients. For the sites we considered, they varied by an order of magnitude, depending on the selected approach. We used the Whitman approach because it is based on test data. But

other analysts could easily justify a different approach.

Finally, the last major subjective decision in the continuum approach is to choose the acceleration time-history record. The SHAKE code²¹ determines the time-history records at any depth. But it is not clear which time history should be used as input for the SSI calculation. Whitman²² emphasizes that the time history should be the one that is experienced near the edge of the interaction stress bulb. But there is no complete agreement on this point; for example, it is common practice to use foundation-level time histories²³ as input for the soil springs.

In general, the lumped mass approach requires the analyst to make numerous judgments, because of a lack of guidelines from proponents of the approach. And it is difficult to make these judgments without a great deal of additional information. Valuable information could be obtained from static or dynamic finite-element methods, e.g., those presented by Johnson et al.¹⁴ But these approaches are still in the developmental stages.

Finite Element Methods

In this section, we discuss the various finite element methods (FEMs) to calculate soil structure interaction.

We start by reviewing the methodology, and then we discuss the advantages and disadvantages of available FEMs.

Finally, we describe the decisions that the FEM analyst is required to make.

SSI INTERACTION METHODOLOGY

The unique feature of the finite element approach is that the dynamic response of the *complete* soil structure system can be obtained. Consequently, several important aspects of calculating SSI for nuclear power plants need to be considered.

First, there are the nonlinear soil properties. Abundant data show that the soil's shear modulus decreases with increased strain, and that the energy dissipated due to damping increases with strain. Because the local soil properties determine the magnitude of the interaction, this softening with strain is clearly crucial to accurate response calculations. In addition, the modeling of damping for finite elements is more complex than simply the incorporation of strain-dependent nonlinearities. First, although damping can result from any of several physical mechanisms, it is believed that soil damping is hysteretic. Often called structural damping, it is proportional to the strain or displacement. Second, because the calculated strains vary from element to element, a damping matrix must be provided for. Most finite element codes include only

viscous damping and allow only for constant damping in each element.

The second important aspect is the nature of the vertical boundaries. The boundaries must be such that energy radiated from the structure does not reflect from them and interfere with the structural response within the time of interest. There are two ways to deal with this. One is to move the boundaries far enough away so that the reflected energy has not returned by close of the calculation. Another way is to employ energy absorbing boundaries, which reduces computer costs.

The third aspect is an accurate high-frequency response. This is a restriction, in designing components and equipment for nuclear power plants, that severely limits the applicability of the available methods.

AVAILABLE SSI METHODS

As discussed earlier, three methods can be used for calculating structural response: mode superposition, time-history direct integration, and complex response. Normal mode codes for calculating SSI with the FEM exist, but they are not in common use. This is because the soil modes are at such low frequencies compared to the frequencies of the structure that it would mean calculating the design response for, perhaps, a hundred modes

to obtain the required high-frequency information. This is prohibitive, given the alternatives discussed below. However, codes in the remaining two methods do exist.

Two representative examples of SSI codes developed from a time-history approach are SOILTIER²⁴ and SLAM.²⁵ The SOILTIER code is a three-dimensional one, with many interesting features. The vertical boundaries are energy absorbing, which reduces the boundary width and computer time. The structure can be modeled with a variety of elements, including rigid floors, space frame members, bracing, and setbacks. However, the soil properties are elastic, and the damping is formulated as Rayleigh damping. The latter point means that the higher modes are strongly damped and, of course, the elasticity ignores soil nonlinearities.

The SLAM code is also very versatile. It has been used extensively in the defense industry. It allows for nonlinear soil properties, separation and sliding at soil structure interfaces, and input excitation either along the base of the model or along the sides of the model. In SLAM, nonlinear formulations have the potential to properly account for hysteretic energy loss, without resorting to an equivalent, viscous damping term. Unfortunately, the currently available soil stress-strain laws in SLAM do

not accurately represent soil behavior.

One final disadvantage of the time-history method is the stability of the time-marching scheme for certain algorithms. This requires additional numerical damping, over and above the actual damping. It has the undesirable effect of damping the higher modes,^{20,26} where, as we discussed earlier, accurate response is usually required.

The most representative code for the complex-response method of calculating structural response is LUSH.²⁷ The principal disadvantage of the complex-response method is that the Fourier components must be superposed to obtain the transient response of the system. This requires the assumption of linear elastic behavior which, as we discussed above, is unreasonable. The LUSH code attempts to circumvent this problem through the equivalent linear method (ELM).²⁸ Although the credibility of the ELM has been questioned, there is no definitive evidence that points to it being invalid. Even though the LUSH code accommodates the nonlinear soil properties in an approximate manner, it has other overriding and significant advantages. First, damping in the complex-response method can be conveniently included as structural, or hysteretic, damping. Second, because the SSI problem is solved in the frequency domain, all the modes are equally

damped at the desired level. As discussed above, these are both significant advantages within the context of our current understanding of damping.

Because of its common usage and its apparent validity, we use the LUSH code in our comparison. However, our use of the code requires the acceptance of several assumptions, whose consequences should be addressed as the subject of a separate study.

SUBJECTIVE DECISIONS

To use the FEM effectively, the analyst must make subjective decisions whose outcome depends on his experience. In other words, the effectiveness of the method might be lessened by the inexperience of the user. Of course, this is true for any calculation, SSI or otherwise. But the question is: Does the method require a very high level of experience or a more general level of experience? We try to answer that question here.

One important decision relates to the mesh refinement and its extent. Because of the filtering caused by discrete meshes, the validity range of the solution will be determined by choosing the right height and width. Another decision is to select the appropriate time step or frequency step desired for the calculation. This outcome also controls the frequency

characteristics of the solution. A third decision concerns the ratio of the lumped mass to the consistent mass. Improper choices of this ratio can result in overestimates or underestimates of the response at resonant frequencies. Perhaps the most difficult and controversial decision is how to make a plane strain model of a three-dimensional structure. Several methods are available.²⁹ None is exact and none is clearly superior; however, the model response errors, in approximating the structure, are less than 30%.^{27,30}

One final decision that the analyst must make applies only to the use of the ELM. The ELM requires the determination of average soil strain during the excitation. And the method selected to accomplish this assumes that the average strain is proportional to the peak strain. This factor of proportionality is required as input and must be determined by the analyst. The analyst could, of course, actually calculate this ratio for the time history of each strain (this capability is programmed into FLUSH³¹), or the analyst could rely on previous experiments and calculations. Available evidence indicates that the factor of proportionality is between 0.5 and 0.7^{21,32} and that the response is relatively insensitive to values in this range. We do not view this as a critical decision.

It is our opinion that all these decisions can be made by any analyst with a general level of experience.

The required judgments do not detract from the overall credibility of appropriate finite element methods.

Comparison Approach

We compare the lumped mass and *finite element methods of calculating SSI* by first assuming a seismic input, site, and structure. We then solve for the structural response with the use of the LUSH²⁷ and SHOCK³³ computer codes, the former to calculate the response of the SSI system and the latter to directly integrate the equations of motion for the structure, with soil springs and dashpots determined by Whitman's methods.¹⁰ The basis for this approach is given in the previous two sections.

To establish the range of validity of these two methods, we solve for the response at four different sites and three different depths of embedment. We use the LUSH code for the finite element calculations and the SHOCK code for the lumped mass calculations, because they satisfy the requirements described earlier. We use the frequency-independent Whitman method to calculate the soil spring parameters, because of its popularity and because of the current NRC needs. However, we recognize that more exact frequency-dependent methods are becoming available, and their assessment should be the subject of a

following project. The rest of this section is devoted to a detailed description of the seismic input for the calculations, the soil sites considered, the structure and the structural model used in the calculations, and finally a discussion of our implementation of the SHOCK and LUSH codes.

SEISMIC INPUT

Directly or indirectly, the seismic input for all the calculations is a surface accelerogram whose response spectrum, when scaled to 1 g, closely follows the response spectrum of NRC's Regulatory Guide (RG) 1.60 (Fig. 2). This accelerogram is used as input for a free field SHAKE²¹ calculation (see next section) to determine the free-field dynamic soil properties and the time-history response at various depths. The bedrock time history is the seismic input for the LUSH calculation. The input for the SHOCK calculation is the time history from an intermediate level whose depth reflects the extent of the anticipated SSI. The details of this latter point are covered in a subsequent section.

Preliminary calculations showed that frequencies over 10 Hz do not

(a)
Response comparison

(b)
Seismic input
time history

Fig. 2. Comparison of seismic input response with response spectrum of NRC's Regulatory Guide 1.60(5% damping).

significantly influence the structural response. Consequently, the surface accelerogram was low-pass filtered at 10 Hz before the deconvolution calculations. Finally, 3.6 sec of zeros were added to the 22.0-sec record to satisfy constraints of the Fourier analysis in the SHAKE and LUSH codes.

SOIL SITES

In our analysis, we consider four sites of two stiffnesses (~ 700 and 1400 fps) and two thicknesses to bedrock (75 and 200 ft). These site parameters cover a wide range of actual situations of interest to the NRC. The sites are composed of clay layers, with the low-strain shear modulus and density increasing with depth. The water table is taken to be at the bedrock level, and Poisson's ratio is taken to be a constant 0.45. The shear modulus variation and the average, acoustic shear-wave velocity V_s (based on the fundamental mode of the site) are summarized for the four sites in Fig.3).

We performed free field calculations on these sites by deconvolving the surface accelerogram with the SHAKE code. We deconvolved to confirm the credibility of the sites through the plausibility of the seismic response, to establish the equivalent linear soil properties, and to determine the response time histories at various levels of the site for input to the SSI calculations.

Originally, we had planned that the peak ground acceleration of the free field would be 0.3 g for both the soft and intermediate sites. However, based on the preliminary SHAKE calculations, it appeared physically impossible for the surface of a 700-fps shear-wave velocity site to respond in accordance with RG 1.60 scaled to 0.3 g. This result is consistent with our intuition and with the experience of other workers in the field. The symptom of the physical impossibility in a SHAKE calculation is the observation that the shear strength of a particular layer tends to go to zero. Also, if the iterations are allowed to continue, the strains become infinite and the SHAKE calculation terminates with an indeterminacy. The indeterminacy can be avoided by rigging the material properties or the calculational scheme. But this is less physically plausible. For example, we found that if hysteretic damping is scaled down from the average values by a factor of 10, a 700-fps site can then transmit sufficiently intense motions to produce the response spectrum of RG 1.60 scaled to 0.3 g at the surface. This scaling of damping makes the soil more elastic; thus, it lowers the effective strain. However, we interpreted the unusually small damping required by this calculation to be an additional argument against using 0.3 g at soft sites.

G = low-strain shear modulus (kips/ft²)

Fig. 3. Shear modulus variation and average, acoustic shear-wave velocity — four selected sites.

Accordingly, we picked a 700-ips site with realistic material properties and determined the most intense surface motions that a deconvolution would permit. This worked out to be 0.1 g associated with RG 1.60. We used this as surface seismic input for the soft sites and, as originally planned, 0.3 g and the RG 1.60 spectrum as the surface seismic input for the intermediate sites.

The soil properties in the free field calculations are initialized with our best estimates. And the SHAKE code was used to iterate with the equivalent linear properties, until the shear modulus and damping factor changed by less than 10%. This generally required four iterations, but never more than seven.

Our deconvolution did not account for an uncertainty in soil properties at the sites. Although important for licensing purposes (Section 3.71 of NCR's Standard Review Plan), this accounting is unnecessary here since our objective is to simply compare two SSI methods.

We summarize the results of these free field calculations in the form of overlaid response spectra of 5% damping for each site (Figs. 4 & 5). The peak accelerations at each level are summarized in Table 4, which is keyed to Fig. 4 & 5. As indicated in the figures and table, the spectral variation with depth has certain

similarities for each site. At increasing periods, the spectral variation with depth decreases; this corresponds to a quasi-static response of the sites. The obvious notch in each set of spectra at 0.1 sec corresponds to the Nyquist frequency associated with the $\Delta t = 0.05$ discreteness of the time histories. Finally, we observe that the shorter the period, the larger the depth variations. In conclusion, we summarize the average shear-wave velocity for each site, based on the dynamic equivalent linear properties in Table 5; this demonstrates the degree to which the sites are softened by the strong motion.

STRUCTURAL MODEL

Since our general objective is to compare two methods for calculating SSI, the structural model is of secondary importance, provided of course that the same model is used for both methods. Nevertheless, to allow for the application of our results beyond the comparison, we selected a model that closely resembles a nuclear reactor.

Models of nuclear reactor buildings are generally very complex. Degrees of freedom, representing the shield building, containment structure, the drywell, shield and pedestal, and the reactor pressure vessel, are excessive

Fig. 4. Depth variations of response spectra (5% damping) -- free field (site thickness = 75 ft; see Table 4 for depth key to curves 1 through 6).

(a)
Site stiffness
700 fps

(b)
Site stiffness
1400 fps

Fig. 5. Depth variations of response spectra (5% damping) — free field (site thickness = 200 f.; see Table 4 for depth key to curves 1 through 6).

Table 4. Free-field response results.

Site thickness ft	Site stiffness fps	Depth ft	Depth key	Peak [*] acceleration g
75	790	0	1	0.10
		20	2	0.68
		30	3	0.67
		40	4	0.69
		50	5	0.61
		75	6	0.54
75	1480	0	1	0.30
		20	2	0.22
		30	3	0.23
		40	4	0.21
		50	5	0.18
		75	6	0.20
200	700	0	1	0.10
		20	2	0.10
		40	3	0.13
		60	4	0.19
		105	5	0.43
		200	6	0.96
200	1400	0	1	0.30
		20	2	0.24
		40	3	0.23
		60	4	0.27
		105	5	0.28
		200	6	0.46

* At 0.01 sec period

Table 5. Acoustic and earthquake average shear-wave velocity.

Site thickness ft	Acoustic shear-wave velocity fps	Earthquake shear-wave velocity fps
75	790	450
75	1480	725
200	700	290
200	1400	570

in number. For example, the GESSAR Mark TV reactor building model has 90 nodes. Because of this complexity, most FEM calculations of SSI are performed on much simpler structural models, with the objective of determining the basemat time history. This time history is then used as input for the basemat of the more complex structural model, to finally obtain floor response spectra, design loads, etc. There are two reasons for this two-step process: (1) experience indicates that SSI is not appreciably influenced by the details of structural models, and (2) the bandwidth of the soil-structure model, with a complex structural model, is so large that it will not fit on most computers. With the advent of the FLUSH computer program and its energy-absorbing boundaries, we may see this two-step process collapse into one. However,

for the purposes of this study, we were constrained to use rather simple structural models in conjunction with the LUSH program.

Our objectives in developing the structural model were to roughly capture the first-mode frequency, the dimensions, and the bearing stress of an actual reactor building. The model was developed in two ways: as a three-dimensional lumped mass model for the SHOCK calculations (Fig. 6, Table 6), and as a plane-strain finite element model (Fig. 7) for the LUSH calculations. We derived the plane strain model by equating the circular foundation geometry, with the width equal to the diameter. We recognize that there are many other ways to derive plane strain representations of real structures²⁹. None of them is exact, and we feel that the alternative methods are no better than the one used in this study.

Table 6. Structural properties for the lumped mass model.

Node No.	Elevation ft	Weight 10 ⁻⁷ lb	Mass moment of inertia 10 ⁻⁹ lb-sec-in.	Element No.	Shear stiffness 10 ⁻¹⁰ lb/in.	Bending stiffness 10 ⁻¹⁵ in.lb/rad	Damping factor fraction
1	5	1.84	6.76	1	7.2	6.1	0.03
2	20	3.66	13.86	2	5.4	4.5	0.03
3	40	3.68	13.86	3	5.4	4.5	0.03
4	60	3.68	13.86	4	5.4	4.5	0.03
5	80	3.68	13.86	5	5.4	4.5	0.03
6	100	3.68	13.86	6	5.4	4.5	0.03
7	120	3.68	13.86	7	5.4	4.5	0.03
8	140	3.68	13.86	8	5.4	4.5	0.03
9	160	3.68	13.86	9	5.4	4.5	0.03
10	180	3.63	13.86	10	5.4	4.5	0.03
11	200	3.68	13.86	11	7.2	6.1	0.03
12	215	1.84	6.76

Fig. 6. Three-dimensional, lumped mass model for the SHOCK calculations (fundamental frequency = 4.5Hz; see Table 6 for materials properties).

We have guaranteed the dynamic equivalence between the lumped mass and the finite element models to make sure that differences in the calculated results are attributable to differences in the methods. Plane-strain finite elements are very stiff in bending, which resulted in the development of a lumped mass, shear-beam structural model for the soil-spring calculations. To test the methods over a range of situations, we assumed

Fig. 7. Plane-strain finite element model for the LUSH calculations (fundamental frequency = 4.4 Hz). Materials properties for each element: density (ρ) = 150 lb/ft³; low-strain shear modulus (G) = 2.95×10^5 kips/ft²; structural damping = 3% critical.

the structural models to be embedded at three depths (0, 20, and 40 ft). The arrangement of these combinations is summarized in Fig. 8.

INPUT FOR LUSH

The important considerations in preparing the LUSH input are the mesh

Fig. 8. Soil structure system.

size and the lateral extent of the model. Our models capitalize on the plane of symmetry. The mesh height is sufficient to pass the highest frequency in the bedrock seismic input. The mesh extends 1000 ft or 16 basemat radii. The adequacy of this lateral extent of the mesh was checked for each of the twelve calculations, but sufficiency was established *a priori* by testing various extents on the softest site. The criterion is that

the response spectrum at the surface boundaries of the model be free field. We limited the number of elements under the structure, as indicated, to keep the bandwidth within the limits allowed by our computer system. The adequacy of this modeling was established through a sensitivity study. For example, in Fig. 9 we compare the superstructure's response spectrum at 5% damping for two finite element runs: one with four elements under the

Fig. 9. Superstructure response spectrum at 5% damping for two finite element runs.

structure, the other with one element. The soil model was layered to be consistent with the SHAKE results. The material properties in each layer were set to the free field values as calculated by the SHAKE code. Each LUSH problem was run for at least three iterations. We found, as others have recently reported,³⁴ that this is more than sufficient to capture the interaction effects.

INPUT FOR SHOCK

The input required for a SHOCK SSI calculation is conceptually simple but very difficult to determine quantitatively. All that is needed are soil spring stiffnesses and dashpots, and an appropriate time history. The model, with soil springs, is depicted in Fig. 10.

The damping coefficients for both rocking and translation consist of two parts: internal damping and radiation damping. The internal damping stems from the stress-strain hysteresis loop that is experienced as the soil undergoes strong excitation. We use the damping factor calculated by the SHAKE code for this. The radiation damping is far more difficult to determine, since it should account for the energy that should be transported away from the vicinity of the structure by the seismic energy radiated from the structure. Based

on several experimental tests for vertical excitation, Whitman¹⁰ recommends the following equations for damping coefficients for horizontal excitation:

$$D_h = \frac{0.31}{\sqrt{\frac{M}{\rho p R^3}}}$$

$$D_\psi = D_{\text{internal}} + \frac{0.1}{\sqrt{\frac{I}{\rho R^5}}} \left(1 + \frac{I}{4 \rho R^5} \right),$$

where ρ = soil density, M = structural mass, I = structural inertia, and R = foundation radius. The coefficients include both internal and radiation damping and, due to the manner in

Fig. 10. Typical soil spring model.

which they are derived from the data, are somewhat conservative. However, since they are the most reliable estimates available, we use them directly in the SHOCK calculations.

Whitman's recommended equations¹⁰ for the translation and rocking stiffness for structures founded at the surface are

$$k_x = \frac{8GR}{(2-u)}$$

$$k_y = \frac{8GR^3}{3(1-u)}$$

where G = shear modulus for soil and u = Poisson's ratio for soil. Because Poisson's ratio is not very sensitive to shear strain, the only term that is difficult to estimate in these equations is G . The problem is that the soil springs represent the effect of the soil surrounding the structure, in response to both the free field and the structural motions, while the only information an analyst has at hand is the static free-field shear modulus. This free field modulus must be corrected for the weight of the structure and the strong motion response of both the soil and structure. Whitman¹⁰ suggests that these dynamic effects can be accounted for by multiplying the measured modulus by a factor of between 0.33 and 1.0. Our analyses showed that the most appropriate factor is 0.33 for the cases considered here. We

based this on the following calculations. First, we performed a static analysis of the soil structure system with the SAP IV code.¹⁵ We initialized the problem with the low strain shear moduli and manually iterated three times with the SAP IV code to obtain a solution that was compatible with the equivalent linear strain. Then we superposed the stresses from the static solution over those from the free-field earthquake response. Modification of the resulting principle shear stress for strain effects resulted in a depth distribution of the shear modulus that includes the effects of earthquake strain, structural weight, and nonlinear soil properties. Then, we averaged the modulus by depth-weighting down to a level that reflects, in our judgment, the deepest expected extent of SSI. We performed these calculations for three different sites. Each time we obtained spring constants that were a factor of three less than the modulus obtained by depth-weight averaging the free field moduli, i.e., consistently the lowest of the multipliers recommended by Whitman. As a result of this agreement, we derived final values for the shear moduli by dividing the depth-weighted average modulus of the free field by three. Table 7 shows the values of these moduli for each site, along with the depth to which the averaging was carried out.

We also followed Whitman's suggestions in modifying spring constants to account for embedment. The basis of Whitman's theory is that the stiffnesses scale up with the increase in structural contact area. Table 8 gives a complete summary of all the spring constants and damping coefficients

used in the SHOCK runs. The indices refer to the structural node numbers (Fig. 6) to which the springs and dashpots are attached. Our calculations show that these soil springs and dashpots result in a flexible base structure with a fundamental mode of 1 to 2 sec. This is a factor of 10

Table 7. Shear moduli used to calculate spring constants.

Site thickness ft	Site stiffness fps	Average shear modulus ₂ kips/ft ²	Depth of averaging ft
75	790	185	75
75	1480	487	75
200	700	40	100
200	1400	250	100

Table 8. Soil spring and dashpot parameters used in the SHOCK calculations.

Soil spring stiffness							Fraction critical damping					
Run	k_1 lb/in.	c_1 in./lb rad	k_2 lb/in.	c_2 in./lb rad	k_3 lb/in.	c_3 in./lb rad	D_1	D_2	D_3	D_4	D_5	D_6
	$\times 10^{-6}$	$\times 10^{-12}$	$\times 10^{-6}$	$\times 10^{-12}$	$\times 10^{-6}$	$\times 10^{-11}$						
7511	4.98	2.63					0.083	0.1				
7521	13.13	6.44					0.083	0.1				
20011	1.0	0.333					0.083	0.1				
20021	6.75	3.56					0.083	0.1				
7512	5.66	2.91	5.66	2.91			0.115	0.1	0.115	0.1		
7522	14.9	7.67	14.9	7.67			0.115	0.1	0.115	0.1		
20012	1.15	0.39	1.15	0.39			0.115	0.1	0.115	0.1		
20022	7.66	3.95	7.66	3.95			0.115	0.1	0.115	0.1		
7513	5.66	2.9	6.35	3.22	5.66	2.9	0.115	0.1	0.14	0.1	0.115	0.1
7523	14.9	7.67	16.74	8.58	14.9	7.67	0.115	0.1	0.14	0.1	0.115	0.1
20013	1.15	0.39	1.29	0.65	1.15	0.39	0.115	0.1	0.14	0.1	0.115	0.1
20023	7.66	3.95	8.59	4.35	7.66	3.95	0.115	0.1	0.14	0.1	0.115	0.1

*Run code: XXX Y Z, where XXX is site thickness (75 or 200 ft), Y is site stiffness (1 = 790 fps, 2 = 1480 fps), and Z is embedment depth (1 = 0 ft, 2 = 20 ft, 3 = 40 ft).

higher than the fixed-base fundamental mode of the structure.

To complete the input for SHOCK, we require an acceleration time history. The appropriate time history is the one observed at the deepest, hypothesized extent of interaction or, for our analysis, at those depths given in Table 7. This is probably the most difficult decision of all and is one that can be checked only through trial and error calculations. It is our experience, however, that selecting time histories from other depths influences only the short period portion of the spectra, especially the peak acceleration. This is consistent with the observed depth variation in response spectra (Figs. 4 and 5).

SUMMARY

We close this section with a summary of the inherent differences between the LUSH and SHOCK approaches to determine structural response from SSI. This should be useful for interpreting the results of our comparative calculations. More detailed discussions of SSI analytic methods are given in Refs. 30 and 34.

Damping

The LUSH code incorporates a frequency-independent, hysteretic damping formulation. The SHOCK code

uses nonproportional, equivalent viscous damping. Within the limits of the equivalent linear method, the LUSH code can be used for calculating material damping effects; also, radiation damping is approximately accounted for through the finite element modeling of the soil deposit. However, for the SHOCK approach, both the material and radiation damping effects are picked by the analyst. But, at present, no guidelines exist for their selection, and the quality of the selection is only as good as the judgment of the analyst.

Soil Moduli

As indicated above, the LUSH code calculates the soil moduli through the equivalent linear method. The selection of soil moduli for the SHOCK approach requires both extra calculations and thoughtful judgment on the part of the analyst.

Excitation

The SHOCK approach requires that the analyst determine an appropriate seismic input. He may derive free-field time histories at several depths with a SHAKE calculation, but he must determine which one is the most appropriate. The only LUSH decision that corresponds to this is the selection of the model depth. For many cases, this depth is the depth

to bedrock, and the decision is therefore determined by the site geometry. Where the appropriate depth is uncertain, a trial-and-error approach will quickly yield a suitable depth. Again, the distinction between the two approaches reduces to one of guidelines for implementation.

Embedded Structures

The embedment effects in the LUSH code are appropriately accounted for, because this code models the soil structure system. On the other hand, the SHOCK theory of embedment has not developed to the point of reliability. At present, there is no agreement as to the best way to calculate the parameters of embedded soil springs. Also, the problem of picking a seismic input referred to above becomes compounded by the analyst having to assume that the same time history is

applied to every embedded spring, or to determine a basis for selecting additional time histories.

Nearby Structures

The structure-soil/structure interaction can be modeled easily with the use of the LUSH code - the only required effort is mesh generation. The SHOCK approach cannot presently account for the effect of nearby structures.

Dimensions

The LUSH code is appropriate for a two-dimensional, plane strain type of model. The SHOCK code is usually used in conjunction with a three-dimensional, lumped mass code. If the LUSH user carefully derives the plane strain model, the errors in the approximation should be acceptably small.

Results and Conclusions

We begin this section by discussing the comparisons between the calculations, including the effects of embedment and site stiffness. We conclude with a discussion of some of the subtleties of the individual calculations.

COMPARISON OF LUSH AND SHOCK RESULTS

The results are summarized in Figs. A-1 through A-6 of Appendix A.

Comparative data of the 5% response spectra at the basement of the structure for LUSH and SHOCK computer codes, at 0, 20, and 40 ft embedments, are given in Figs A-1 through A-3. Corresponding data for the superstructure (uppermost node) are shown in Figs. A-4 through A-6.

Note that because of the stiffness of the structure relative to the soil, the SHOCK-calculated motion of the

basemat is directly related to the translational spring, while that of the superstructure is related to both soil springs, but mostly to the rocking spring. To assess each of these spring constants, we first analyze the basemat comparison, then the superstructure one. Note that we expect the lumped mass (SHOCK) results to be conservative relative to the finite element (LUSH) results, largely because of the selection of conservative damping values.

A first observation confirms that the SHOCK results are, in general, often conservative, particularly for the unembedded case; however, there are notable exceptions. First, the soft 200-ft site for all depths of embedment indicates exactly the opposite, for all periods. This site, for purposes of calculating soil spring constants, was very anomalous. Although the site behaved in a predictable manner when excited by acoustic strain motions, it produced dramatic layering in response to earthquake strains, as indicated by the variation of shear modulus and response spectra with depth. Consequently, the depth-weighted, average shear modulus for the site is less meaningful, because of the large variations about this mean value with increasing depth. The resultant soil spring constants cannot accurately capture the actual layering at the

site and, therefore, result in poor estimates of the interaction effects. We believe that if more appropriate spring constants (to account for the strong layering)* were used, the results for this site would be consistent with those from the other sites (i.e., indicating that the SHOCK approach is conservative). Therefore, we consider this site as being totally beyond the capability of the SHOCK approach.

Other exceptions to our generalization appear to be related to embedment depth. With increasing embedment, spikes appear at the 1 to 2 sec period in the LUSH results that are not observed in the SHOCK results. The inability of the embedment springs to model the effects of embedment as they are modeled in the LUSH code accounts for this lack in the SHOCK results. We conclude that embedment effects cannot be modeled in the SHOCK approach by the methods described above.

A final point of the basemat motion is that the stiffness of the structure is so much greater than that of the soil that the structure can be considered rigid. As a result, the basemat translational motion is largely a function of the soil translation spring and its dashpot. We

*This is, unfortunately, beyond the state of the art.

can use this observation to assess the accuracy of the spring constant and its dashpot in the following way. If the locations of the spectral peaks are different in the LUSH and SHOCK calculations, then we can say that it is likely the soil spring constant is in error. If, however, the differences are in the height of the spectral peaks, then the error is probably in the damping coefficient. Finally, if there are differences in the peak acceleration, it must be the result of a combination of errors in the spring constant and damping coefficient. Applying these observations to the surface-founded cases, most of the differences (excepting 20011)* can be accounted for by adjustments to the damping coefficient. However, for increasing depths of embedment, we observe that increasing adjustments to the translational spring constants are required. This latter point is consistent with the conclusion given above.

There is another possible explanation for the low period difference in results. The most uncertain assumption in the LUSH approach is the equivalent linear method. The apparent consequence of this assumption is excessive damping in the higher frequencies.³⁶ This could account for

the fact that the LUSH results are often lower in spectral amplitude than the SHOCK results for the higher frequencies.

In the superstructure results, it is useful to note that because of the structure stiffness, the superstructure motion is most sensitive to the rocking spring and dashpot (although the response does include some effects of the translational spring). The superstructure results are similar to the corresponding basemat results — with one important exception. Apart from the anomalous 200-ft soft site, the LUSH spectral accelerations exceed those of SHOCK at longer periods. Because these FEM amplifications were not observed in the basemat motions, they must be the result of one of the following factors: (1) different structural models in the LUSH and SHOCK codes, (2) long-period spectral amplification due to the FEM box effect,³⁷ or (3) inconsistent rocking springs in the SHOCK model. It cannot be a difference in structural models, because we have verified structural equivalence and besides, the spectral amplifications are at periods far above those of any structural modes. Also, we doubt that the amplifications are caused by an insufficient soil model size; the size is such that we generally recover free field motions at the edge of the model and, where we have been unable to recover them,

* Site thickness = 200 ft; stiffness = 700 fps; embedment depth = 0 ft.

the differences are in the high frequency range and not in the long period. Finally, there is the possibility that the rocking soil spring is incompatible with the "effective" LUSH soil spring. Based on our experience, if there is an inconsistency in the spring constant, it must be moderately large (25-50%), because the structural response is not critically sensitive to the magnitude of this spring constant. On the other hand, the problem could be that of neglecting the spring dependency on frequency. This, we feel, is most likely.

In summary, the basemat and super-structure spectral accelerations differ significantly for the SHOCK and LUSH calculations - sometimes by as much as an order of magnitude. We believe that the differences result principally from neglecting the frequency dependency of the soil spring constants. However, there are other possible explanations, which should be addressed in future work. It is very important to note that the largest differences between the LUSH and SHOCK calculations are at the longer (> 1 sec) periods. This may have large practical implications. For example, if it can be shown that these frequencies are outside the range of interest for nuclear power plants, then a more realistic comparison should be made over a more

limited frequency range. It is possible that good agreement could be obtained simply by using more sophisticated techniques in selecting soil spring constants without regard for the frequency dependency in this higher frequency range.

EMBEDMENT AND SITE STIFFNESS EFFECTS

The embedment effects are given in Figs. B-1 and B-2 (Appendix B). The results, given for both SHOCK and LUSH basemat calculations, all display an intuitively obvious trend. As embedment depths increase, the peaks in the response spectra move to higher frequencies, and this effect is more marked for the softer sites. The stiff 200-ft site, for example, shows negligible effects of embedment, while the soft 75-ft site shows significant frequency shifts.

Representative plots of basemat response spectra, showing the effect of site stiffness, are given in Figs. C-1 and C-2 (Appendix C). Again, the overall results are largely intuitive and are useful in providing a feel for the magnitude of the effect. In every case for both methods, the stiffer site experiences larger spectral accelerations. The spectral peaks occur at resonances in the soil, and at these peaks the differences approach a factor of 10.

We conclude from these results that, with regard to SSI, there is a very big difference between a 700-fps site and a 1400-fps site: interaction effects dominate at the former, while at the latter they are almost negligible. We can make this last point clearer with the results from an interesting calculation.

An illustrative test of the significance of SSI in the FEM approach can be obtained by comparing the iterated, equivalent linear solution with the solution that assumes the soil is elastic but initialized with the free field SHAKE results. Such a comparison, for a stiff site, is illustrated in Fig. 11. The figure shows gross

Fig. 11. Comparison of iterated, equivalent linear solution with uniterated solution - soil structure interaction in the FEM approach (site thickness = 200 ft; stiffness = 1400 fps; depth = 0 ft).

similarities between the two spectra, particularly in the more significant higher frequency range (< 1 sec period). This similarity suggests that the secondary interaction effects³⁴ are relatively small and that the site, therefore, is sufficiently stiff to minimize soil structure interaction.

ADEQUACY OF LUSH MODEL

As we discussed in an earlier section, it is important that the vertical boundaries be located far enough away so that energy does not reflect back to the structure. Our criterion in this regard is that the response spectra calculated along these vertical boundaries must agree with the free field spectra. Other criteria are often suggested,³⁸ but we feel that this is the most realistic one, even though it results in models of very large lateral extent. We show an example of this agreement in Fig. 12. Except for the softest site, the agreement for all other cases is similar. The free field motions at this site suffered significant high-frequency attenuation, as can be seen in Fig. 13. Unfortunately, the lateral extent of the mesh could not be extended without compromising other aspects of the solution (duration of the time history, etc.), since the problem already used all the available

SCM and LCM of a CDC 7600 computer. However, we do not believe that these differences in free field spectra between the LUSH and SHAKE calculations significantly influence our structural response, because the structure tunes to the soil resonances, which are at a period of 1 to 2 sec.

Finally, in Fig. 14, we show how the response spectra typically vary along the surface. It is clear that motions at the edge of the mesh are significantly modified along the surface.

CONCLUSIONS

Our objective in this study has been to compare two approaches for calculating soil structure interaction: finite element and lumped mass. To this end, we have calculated the SSI response in 12 representative situations using the LUSH computer code and SHOCK computer code in conjunction with Whitman soil springs and dashpots. The results from the lumped mass method are generally conservative relative to those of the finite element method. Our analysis suggests that a closer agreement between the two methods could be obtained, if frequency-dependent soil springs and dashpots were used in the lumped mass calculations, particularly for the embedded cases. Regardless of whether

(a)

Site thickness = 75 ft
stiffness = 700 fps
depth = 40 ft

(b)

Site thickness = 200 ft
stiffness = 1400 fps
depth = 40 ft

Fig. 12. Comparison of LUSH and SHAKE free field surface spectra.

Fig. 13. High-frequency attenuation of free field motions - LUSH and SHAKE.

or not frequency-dependent soil parameters are used, we are concerned with the total lack of guidelines for implementing the lumped mass approach. The absence of guidelines from researchers, which we believe is caused by the variety of soil spring approaches currently being proposed, is also a source of concern for the industry. For example, "...errors in

the assessment of spring constants and damping values can lead to large differences in computed response and guides to judgment in this respect are urgently needed."³⁹ In our judgment, this lack of suitable guidelines for a lumped mass approach makes the finite element method a generally superior one for calculating soil structure interaction.

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Fig. 14. Surface motion in FEM calculations.

formulation in the SHOCK code, and in lumped mass codes in general, helped us to better appreciate the significance of damping. We received many valuable suggestions for improving the report from Don Arthur, Don Bernreuter, and Paul Smith who reviewed

preliminary drafts. We received several useful comments, as well as some thought-provoking ideas, from Professor R. V. Whitman; his interest is deeply appreciated. Finally we wish to thank Frank Tokarz for his advice and support throughout this project.

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Appendix A

Response Spectra Comparison Data – Structure Basemat and Superstructure

(a) Site thickness = 75 ft
stiffness = 700 fps

(b) Site thickness = 75 ft
stiffness = 1400 fps

(c) Site thickness = 200 ft
stiffness = 700 fps

(d) Site thickness = 200 ft
stiffness = 1400 fps

Fig. A-1. Comparison of 5% response spectra at the structure basemat for LUSH and SHOCK computer codes — 0 ft embedment.

(a) Site thickness = 75 ft
stiffness = 700 fps

(b) Site thickness = 75ft
stiffness = 1400 fps

(c) Site thickness = 200 ft
stiffness = 700 fps

(d) Site thickness = 200 ft
stiffness = 1400 fps

Fig. A-2. Comparison of 5% response spectra at the structure basemat for LUSH and SHOCK computer codes - 20 ft embedment.

(a) Site thickness = 75 ft
stiffness = 700 fps

(b) Site thickness = 75 ft
stiffness = 1400 fps

(c) Site thickness = 200 ft
stiffness = 700 fps

(d) Site thickness = 200 ft
stiffness = 1400 fps

Fig. A-3. Comparison of 5% response spectra at the structure basemat for LUSH and SHOCK computer codes - 40 ft embedment.

(a) Site thickness = 75 ft
stiffness = 700 fps

(b) Site thickness = 75 ft
stiffness = 1400 fps

(c) Site thickness = 200 ft
stiffness = 700 fps

(d) Site thickness = 200 ft
stiffness = 1400 fps

Fig. A-4. Comparison of 5% response spectra at the superstructure for LUSH and SHOCK computer codes — 0 ft embedment.

(a) Site thickness = 75 ft
stiffness = 700 fps

(b) Site thickness = 75 ft
stiffness = 1400 fps

(c) Site thickness = 200 ft
stiffness = 700 fps

(d) Site thickness = 200 ft
stiffness = 1400 fps

Fig. A-5. Comparison of 5% response spectra at the superstructure for LUSH and SHOCK computer codes - 20 ft embedment.

(a) Site thickness = 75 ft
stiffness = 700 fps

(b) Site thickness = 75 ft
stiffness = 1400 fps

(c) Site thickness = 200 ft
stiffness = 700 fps

(d) Site thickness = 200 ft
stiffness = 1400 fps

Fig. A-6. Comparison of 5% response spectra at the superstructure for LUSH and SHOCK computer codes - 40 ft embedment.

Appendix B

Embedment Effects Data –
SHOCKS and LUSH Basemats

(a) LUSH (finite element)
Site stiffness = 700 fps

(b) SHOCK (lumped mass)
Site stiffness = 700 fps

(c) LUSH (finite element)
Site stiffness = 1400 fps

(d) SHOCK (lumped mass)
Site stiffness = 1400 fps

Fig. B-1. Comparison of embedment effects, resulting from 5% response spectra at the structure basemat, for LUSH and SHOCK computer codes - 75 ft site thickness.

(a) LUSH (finite element)
Site stiffness = 700 fps

(b) SHOCK (lumped mass)
Site stiffness = 700 fps

(c) LUSH (finite element)
Site stiffness = 1400 fps

(d) SHOCK (lumped mass)
Site stiffness = 1400 fps

Fig. B-2. Comparison of embedment effects, resulting from 5% response spectra at the structure basemat, for LUSH and SHOCK computer codes - 200 ft site thickness.

Appendix C

Stiffness Effects Data – SHOCKS and LUSH Basemats

(a) LUSH (finite element)
Embedment = 0 ft
Site thickness = 75 ft

(b) SHOCK (lumped mass)
Embedment = 0 ft
Site thickness = 75 ft

(c) LUSH (finite element)
Embedment = 20 ft
Site thickness = 75 ft

(d) SHOCK (lumped mass)
Embedment = 20 ft
Site thickness = 75 ft

Fig. C-1. Comparison of stiffness effects, resulting from 5% response spectra at the structure basemat, for LUSH and SHOCK computer codes - 75 ft site thickness.

(a) LUSH (finite element)
 Embedment = 40 ft
 Site thickness = 200 ft

(b) SHOCK (lumped mass)
 Embedment = 40 ft
 Site thickness = 200 ft

Fig. C-2. Comparison of stiffness effects, resulting from 5% response spectra at the structure basemat, for LUSH and SHOCK computer codes - 200 ft site thickness.

Appendix D

SI Conversion Table

Quantity	From	To convert	
		To	Multiply by
Distance	feet	meters	0.3048
Velocity	feet/sec	meters/sec	0.3048
Shear moduli	kips/ft ²	newtons/meter ²	47880.0
Stiffness	1b /in.	newton/meter	175.12
	in.-lb/radian	newton-meter/radian	0.1130
Mass moment of inertia	1b-sec ² -in.	newton-sec ² -meter	0.1130
Density	1b/ft ³	kilogram/m ³	16.02