THE EFFECT OF FOUNDATION SOILS ON SEISMIC RESPONSE OF STRUCTURES

by

W.D. Liam Finn(I), J.J. Emery(II) and R.B. Reimer(III)

SYNOPSIS

The dynamic response of a 10 storey building on a 200 ft deep foundation layer was determined for sinusoidal excitations and a scaled El Centro earthquake record. Both coupled and uncoupled systems were considered. It was found that the responses were very similar for the coupled and uncoupled systems. The greatest magnification in response occurred in all cases when the fundamental periods of the building and foundation coincided. Results clearly show that the response of a structure to an earthquake depends on the dynamic characteristics of both the structure and the ground.

INTRODUCTION

Building codes commonly consider that the dynamic response of a structure depends only on the nature of the arriving seismic waves and on the dynamic characteristics of the structure. The structure is often assumed to rest on a rigid base and some 'appropriate' ground motion is fed into it at the base level, or else the dynamic response is deduced from response spectra of the same 'appropriate' ground motion. However, the dynamic response is in fact also affected by the properties of the earth's crust, by local foundation soil conditions and by coupling between the structure and its foundation. Furthermore, the ground motion used in the dynamic analysis is often not typical of the particular site. The ground motion adopted in most codes is for rather 'firm' soils with short fundamental periods. For 'less firm' soils, most codes suggest empirical increases in the computed dynamic loads regardless of the nature of the structure, one of the few exceptions being the Chilean Code.

Studies of earthquake damage to structures such as those by Duke (4)(IV) and especially Ohsaki (9) clearly indicate that the nature of the structures must be considered when assessing the effect of foundation soils on dynamic structural response. For example, during the Kanto earthquake of 1923 more brick buildings (Godowns) were damaged on firm soils than on soft soils. Generally, the opposite trend was observed for wooden structures which are far more flexible than rigid brick buildings. This suggests that structural resonance and damage occurred when a structure's natural period approached the

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(I) Professor and Dean of Faculty of Applied Science, University of British Columbia, Vancouver, B.C.

(II) Research Associate, Faculty of Applied Science, University of British Columbia, Vancouver, B.C.

(III) Graduate Student, Department of Civil Engineering, University of California, Berkeley, California, U.S.A.

(IV) See the list of references at the end of the paper.

predominant period of the ground. The height of the buildings (the highest was only 3 storeys) had little effect on the damage to brick buildings (9). In Mexico City, during the 1957 earthquake, there was a comparatively high damage rate to reinforced concrete buildings of 13 to 16 storeys founded on the deep lacustrine deposits of the Valley of Mexico. On the other hand, in Skopje, where the soils are firmer (a sandy gravel layer about 35 ft thick on bedrock), most brick buildings of less than 4 storeys were heavily damaged but reinforced concrete buildings of 13 to 14 storeys suffered only minor damage (9). Clearly the damage due to shaking on different foundation soils depends not on the soil conditions independently, as most codes suggest, but on the relation between the dynamic characteristics of the structure and foundation soils.

The effect of foundation characteristics on structural response was considered explicitly by Finn and Khanna (5) in 1966 for dams on elastic foundations. They showed that the important variable controlling the response of the coupled system of dam and foundation was the ratio of the fundamental period of the coupled system to the predominant period of the earthquake excitation. When this ratio approached unity a quasi-resonant state was reached leading to large displacements and stresses.

The interelationship of the various factors that determine the dynamic response of a structure during an earthquake can be expressed in a general form as suggested by Ohsaki (9):

$$R(t) = f\left[E(t),\ G(t),\ F(t),\ S(t)\right] \tag{1}$$

where

R(t) is the response of the structure during an earthquake,

f is a functional,

E(t) is a function representing the epicentral movement,

G(t) is a function representing the properties of the earth's crust,

F(t) is a function representing the foundation soil conditions,

and S(t) is a function representing the dynamic properties of the structure.

In this equation, none of the functions are independent and they interact with each other. This paper is primarily concerned with the foundation soil conditions F(t) and the interaction between F(t) and the dynamic properties of the structure, S(t). It is considered that the bedrock record below the foundation site is known and this would include the interaction of E(t) and G(t). However, deformability of the bedrock is not considered and it must be recognised that there is some interaction between G(t) and F(t).

ANALYTICAL APPROACHES TO SOIL STRUCTURE INTERACTION

Two general approaches are being used in analytical investigations of the soil-structure interaction problem (12). In one approach, the foundation is idealised as a linearly elastic half-space or as a semi-infinite layer upon which a structure rests. Flexibility and equivalent viscous damping influence functions at the soil-structure interface are determined to account for the compliance of the foundation material and for the loss of energy in the system due to transmission of elastic waves into the continuum. A suitably discretised structure can then be analysed including the coupled foundation effects by imposing an excitation at the base of the semi-infinite layer or at the surface of the half-space depending on the foundation idealisation used. This approach has been used by Chopra and Perumalswami (2), Parmalee (10) and Rainer (11).

The continuous elastic foundation technique is limited principally by the availability of analytical solutions for the dynamic response of the half-space or semi-infinite layer under a time-varying surface load. These solutions are required in order to evaluate the flexibility and damping influence functions. The most commonly used solution is that due to Bycroft (1) for a rigid circular plate with three degrees of freedom. Besides the usual assumption of an ideal linearly elastic homogeneous isotropic material, the distribution of pressure beneath the plate must be assumed in order to satisfy the mixed boundary conditions at the interface of the continuum and structure.

The second approach utilises the finite element method of analysis which is based on a complete discretisation of the soil-structure system. This technique was used by Finn and Khanna (5) in the 1966 study and by Finn and Reimer in a recent study (6), and is also used to obtain all the results presented herein. For a description of the finite element method of dynamic analysis see Clough and Chopra (3) or Finn and Khanna (5).

SYSTEMS EXAMINED

The dynamic structural response of a 10 storey building on a foundation layer 200 ft in depth was examined for various foundation elastic moduli. In each case, the structural response for the input acceleration record was computed for three different assumptions: (1) the structure is on bedrock or a rigid foundation, (2) the structure and foundation layer are coupled, and (3) the structure and foundation layer are uncoupled. These three cases are shown schematically in Figures 1a, 1b and 1c respectively. For the structure on a rigid foundation, the acceleration is fed into the base of the structure. This would correspond to the direct use of an acceleration record measured on bedrock. In the coupled system, the acceleration record is fed directly into the base of the combined foundation layer and structure system and the acceleration response or base shear response in the building is computed. This would appear to be the most rational approach to the seismic analysis of structures on flexible foundations. In the uncoupled system, the acceleration response at the surface of the foundation layer was first computed for the acceleration record fed into the base of the foundation layer alone. This acceleration response record was then applied to the base of the structure and the structural responses computed, assuming the structure to have a rigid base.

The latter method is commonly used in seismic analysis for the following reasons: (1) only at or near ground surface acceleration records from earthquakes have been available for analysis, and (2) until recently, no practical methods for computing the response of the coupled structure-foundation system have been available. Thus, designing a structure to resist the elastic forces induced by the Mexican earthquake of 28th July, 1957, is an example of an uncoupled analysis since the record was obtained at Alameda Park, Mexico City, on the free surface of about 1600 ft of very soft soil (13). Clearly, it is important to determine if such an uncoupled approach leads to a reasonable analysis.

In a preliminary parametric study of structure foundation interaction, sinusoidal input accelerations of 1 g amplitude and of different frequencies are used in order to more conveniently characterise the nature of the excitation. Then, the N.S. component of the El Centro, California, earthquake of 18th May, 1940, scaled down to 0.2 g is adopted as a typical ground motion for the examination of the various cases. This earthquake record was obtained near the free surface of a very stiff deposit 100 ft deep and is typical of several strong ground motion records measured in the United States (7).

FINITE ELEMENT IDEALISATION

The finite element representation of the 10 storey building on 200 ft deep foundation is shown in Figure 2 and the properties of the structure and the various foundation moduli are given in Table 1. The foundation is of essentially infinite lateral dimension and the centre line of the building is a plane of symmetry with respect to the foundation mesh. The lateral extent of the mesh with respect to its depth is an important consideration when applying the finite element method to an assumed semi-infinite layer. Effects of fixed edges located a finite distance from the structure on the response of the structure must be minimised.

For the foundation layer considered here, the fixed vertical boundaries of the semi-infinite surface layer were located approximately 30 times the layer thickness on each side of the centre line. Analyses of cases with these dimensions (6) showed no significant variation from free field accelerations at a distance of about 3000 ft from the centre line. This would appear to be in agreement with results obtained by Idriss (8) for a finite element analysis of the seismic response of earthbanks.

Plane strain conditions were assumed to exist in the soil and constant strain triangular elements were used throughout. The soil was considered to be linear elastic with a range of elastic moduli corresponding to a shear wave velocity, V_S, of from 370 fps (E_F=10000 psi) to 1170 fps (E_F=100000 psi). This range of shear wave velocities for earth materials corresponds quite closely to the range determined by various shaking tests on earth dams (14) and values of shear wave velocity versus N values for soils in Japan given by Ohsaki (9). The other properties of the foundation soil were kept constant and are shown in Table 1a. The mass distribution was obtained by lumping one third the mass of each triangle at each of its nodes.

The properties of the building given in Table 1b are for a 100 ft wide building. It was assumed that a 1 ft 'slice' of this building could be assumed to act on a 1 ft wide strip of the foundation layer and the storey stiffnesses and weights used are $^1/100$ of those listed in Table 1b. Axial deformations in the columns and girders of the building were neglected and the base columns were assumed to be rigidly fixed to the foundation layer. Rotational degrees of freedom were allowed at the structural nodes. The mass distribution for the building was concentrated at each floor level.

Only horizontal accelerations were considered in the present analyses. However, recent records such as those for the Los Angeles earthquake indicate that vertical accelerations are also very important in determining the dynamic response of structures. Combined horizontal and vertical accelerations will be included in future studies. Elastic behaviour was assumed throughout the present studies. The general analysis, however, is not limited in any way to linear elastic behaviour of ground or structure.

CHARACTERISTICS OF SYSTEM PERIODS AND MODE SHAPES

For the foundation layers (F), the eigenvalues and, hence, the periods are very close and are often paired as indicated by the first 15 frequencies and periods for the foundation layer with modulus 50000 psi (F_{50}) given in Table 2. The first two mode shapes corresponding to the first two periods for F_{50} have been plotted at the surface of the layer in Figure 3 and it can be seen that these correspond to a symmetric and antisymmetric pair of eigenvectors. Also, the sinusoidal shape of these modes at the surface is very close to that anticipated from analytical solutions for the mode shapes of semi-infinite layers. It would appear that this bunching of periods in pairs and the resulting symmetric and antisymmetric mode shapes is typical for long soil layer problems.

In Table 3, the first three periods for each of the systems is given. The effect of increasing stiffness on the first period of the foundation layers is clear and the decrease in the first period is in proportion to the square root of the inverse ratio of the elastic moduli as anticipated. Also, the period of each foundation layer has been determined from the expression for the natural frequencies, ω_n, for the free vibration of a semi-infinite layer with only shear deformations allowed:

$$\omega_n = \frac{(2n-1)\pi}{2H} V_s \qquad (2)$$

where V_s is the shear wave velocity and H is the depth of the soil layer. There is quite close agreement between these calculated periods and those determined from the finite element analysis, although ω_n determined from Equation (2) considers only pure shear whereas the finite solution considers general two-dimensional motion.

For each coupled foundation and structure case (FS), the fundamental period of the structure alone (T_S) tends to show up as the period of one of the general mode shapes of the combined system (T_F). This is indicated in Table 3 for cases $F_{26}S$, $F_{29}S$, $F_{50}S$ and $F_{100}S$. However, for case $F_{10}S$ where the foundation modulus E_F was quite low at 10000 psi and T_F was quite high, the fundamental structural mode was the 27th mode of the coupled foundation and structure case. For this reason, 30 modes were used in the investigation of case $F_{10}S$. Cases $F_{26}S$ and $F_{29}S$ were selected to give systems with T_S approximately equal to T_F as indicated in Table 3. There was a small change in T_S for each coupled case, but it would appear that coupling had little effect on the mode shapes and periods for the building and foundation used. Of course, as earlier studies indicated (6), the effect of coupling on periods and mode shapes for very massive structures such as earth dams is far more pronounced.

RESULTS OF THE PARAMETRIC STUDY

For the parametric study, each system was subjected to a sinusoidal acceleration record of amplitude 1 g. The frequency of the acceleration record was varied so that response curves of each system could be developed. In

Figure 4, the response of the coupled system $F_{26}S$ and uncoupled system $F_{26}\rightarrow S$ for the foundation layer with modulus E_F equal to 26000 psi are shown. For this case, the ratio of the fundamental period of the foundation layer to the fundamental period of the coupled system, T_F/T_C is 1.00 and the ratio of the fundamental period of the structure to the fundamental period of the coupled system, T_S/T_C is 0.95. Thus, case $F_{26}S$ would probably represent the most severe case from an earthquake engineering viewpoint, particularly if the predominant period of the earthquake T_E coincided with T_C. The magnification during shaking was taken as the ratio of the maximum base shear force in the building V to the weight of the building W. A check at very low frequencies gave a magnification of unity as anticipated. For the structure, S, on the rigid foundation, the magnification at quasi-resonance (referred to as resonance hereafter) was about 7 when T_S/T_E was slightly greater than 1 as would be expected for a system with 5% damping. However, for the coupled system $F_{26}S$ the magnification was about 52 and for the uncoupled system $F_{26}\rightarrow S$ the magnification was slightly greater at about 53. For all of the cases examined there was very little difference in the response for uncoupled and coupled systems.

Cases $F_{26}S$ and $F_{26}\rightarrow S$ were the most severe examined since resonance of foundation and structure was inevitable when the period of the sinusoidal excitation coincided with the fundamental period of the structure and foundation. The magnification was about 9 through the foundation layer and 6 through the structure for a total of about 52 or 53 for the coupled or uncoupled systems. Some of the other cases examined are shown on Figure 5 for E_F=10000 psi, E_F=26000 psi, E_F=100000 psi and the structure on a rigid foundation. Only the coupled systems are shown since the difference between coupled and uncoupled response was quite small for all the cases examined. The increase and then decrease in response as the foundation modulus E_F is increased shows up clearly. For $F_{10}S$, the fundamental structural mode shows up as a bulge to the right of the fundamental coupled mode. The response for the stiff system $F_{100}S$ is approaching that for the structure on a rigid foundation.

To check whether horizontal deformations of the foundation layer between the two base columns of the building were introducing shear forces, the two foundation nodes to which the building is rigidly fixed were given the same horizontal degree of freedom. This corresponds to a building with a rigid floor slab. No difference in the results was found for any case when this rigid floor slab was introduced. Horizontal deformation of the foundation layer between the base columns was not introducing shear forces, and the large shear forces were due solely to the horizontal movement of the systems.

The maximum response of each system at resonance is shown on Figure 6. Except near resonance, the maximum response occurred in a few cycles of the sinusoidal excitation long before the steady-state response was reached. However, near resonance, the response kept increasing in the lightly damped systems and the maximum response at resonance was taken in the first 10 seconds of excitation. By this time the rate of increase was small and, after 20 seconds, was only about 5% greater. The magnification for each case has been taken as the ratio of the maximum base shear force at resonance for the system V, to the maximum base shear force at resonance for the structure on a rigid foundation V_S. On this composite figure, the importance of the ratio of the fundamental period of the structure, T_S, to the fundamental period of the foundation, T_F, in determining the response again shows up very clearly. For $F_{26}S$ and $F_{26}\rightarrow S$, when T_S/T_F is approximately unity, the magnification is greatest as indicated. The magnification decreases markedly below and above T_S/T_F = 1. It can also be seen that uncoupling has little effect on the response.

The parametric portion of the study has indicated two major points:

(1) For the building and foundation layers examined, there was very little difference between the coupled and uncoupled response for the sinusoidal excitations. Thus, the effects of interaction between the structure and soil were quite small and would not have to be considered in describing the response of such a system. Since the building and foundation examined have a very reasonable range of properties, it would appear that soil-structure interaction problems can be uncoupled except in the case of massive structures such as dams (6) or very heavy buildings on relatively shallow layers.

(2) The greatest magnification in response to the sinusoidal excitations occur when the fundamental periods of the building and foundation layer coincide. The damage due to shaking on different foundation soils does not depend on the soil or structural properties alone, but on the relationship between the dynamic characteristics of the foundation and structure.

STUDIES USING SCALED EL CENTRO

The response of each of the systems was also determined for the first 10 seconds of the N.S. component of the El Centro, California, earthquake of 18th May, 1940. The earthquake record was scaled down to give a maximum amplitude of 0.20 g which would correspond to the amplitude used in several design studies in British Columbia. Also, it is felt that this might be closer to the maximum amplitude of the actual record at bedrock since the available record was obtained near the free surface of a very stiff deposit 100 ft deep (13). The El Centro earthquake has a similar spectrum to the Taft, California and Olympia, Washington, earthquakes and is often considered representative of strong motion records measured in the United States (7). The predominant period of El Centro, TELC, was taken as 0.45 sec from a consideration of the acceleration spectra. The results of this study are summarised in Figure 7 which shows the variation of maximum base shear force in the building and maximum ground acceleration for various foundation conditions. It may be noted that the maximum ground acceleration and the maximum base shear force occur under different foundation conditions. The former occurs when the fundamental period of the foundation layer is close to the predominant period of the earthquake arriving at bedrock level, T_F/T_{ELC}. The maximum base shear force occurs when the fundamental period of the structure is equal to the fundamental period of the foundation ($T_S/T_F \simeq 1$) which is located in Figure 7 where T_F/T_{ELC} is approximately 3.

A recent example of this effect is cited by Tezcan (15) in discussing the collapse of buildings at the Fiat automobile factory in Bursa, Turkey, during the 1970 Gediz earthquake. From spectrum curves of the after-shocks, the dominant period of the foundation soil (T_F) was found to be 1.20 sec. The natural periods of the partially collapsed buildings (T_S) were about 1.25 sec. Although the ground acceleration at Bursa, 135 km. from the epicentre, was only 0.04 g, because of the coincidence of building and foundation period the ground acceleration was magnified 4 to 5 times for a damping ratio of 5%. Since the buildings were designed for 0.06 g, collapse due to resonance was inevitable.

CONCLUSIONS

Two important conclusions may be tentatively drawn from this preliminary study:

(1) For most structures, an uncoupled analysis using a surface record is adequate to determine the seismic response of the structure.

(2) Considerable amplification may be anticipated when the fundamental periods of the structure and foundation soil are close to one another. Structures whose fundamental period approximates the ground period may be subjected to damage at large distances from the epicentre of an earthquake, even though the arriving wave may have small acceleration amplitudes as in the example of the Fiat buildings discussed earlier.

Other studies have shown that massive structures such as earth dams or very heavy structures on shallow foundation layers respond differently. For these structures it appears advisable and necessary to use a coupled analysis.

ACKNOWLEDGEMENTS

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TABLE 1

(a)

PROPERTIES OF THE FOUNDATION LAYERS

200 ft Deep

Case	Elastic Modulus psi	Unit Weight lb/ft^3	Poisson's Ratio ν	% Critical Damping in Each Mode	Shear Wave Velocity, V_s fps
F_{10}	10000	130	0.3	5	370
F_{26}	26000	130	0.3	5	590
F_{29}	29000	130	0.3	5	630
F_{50}	50000	130	0.3	5	825
F_{100}	100000	130	0.3	5	1170

(b)

BUILDING PROPERTIES

120 ft High, 100 ft by 37.6 ft in Plan
5% Critical Damping in Each Mode

Storey Number (from ground level up)	Total Flexural Rigidity EI (lb-in^2) per Storey		Storey Weight kips
	Columns	Girders	
1 to 4	24.84×10^{10}	48.60×10^{10}	200
5 to 7	12.42×10^{10}	48.60×10^{10}	200
8 to 10	6.21×10^{10}	37.30×10^{10}	200

TABLE 2

FIRST 15 FREQUENCIES AND PERIODS FOR FOUNDATION LAYER F_{50}

$E_F = 50000$ psi

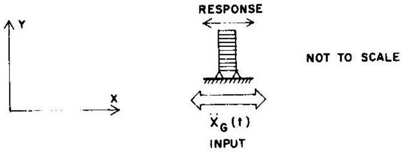
Mode No.	Frequency cps	Period sec
1	0.941056)	1.062636)
2	0.941089)	1.062598)
3	0.976146)	1.024437)
4	0.976471)	1.024096)
5	1.012389	0.987763
6	1.016451	0.983815
7	1.030493	0.970410
8	1.059133	0.944169
9	1.077509	0.928066
10	1.098458	0.910367
11	1.137873	0.878833
12	1.167006	0.856894
13	1.190839	0.839744
14	1.228252	0.814165
15	1.248303	0.801087

TABLE 3

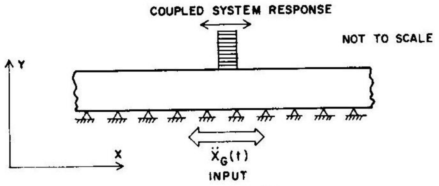
FIRST THREE PERIODS FOR EACH SYSTEM

Case	Description	Finite Element Solution			Analytical Solution
		1st Period sec	2nd Period sec	3rd Period sec	1st Period
S	Structure Alone	1.396819	0.514648	0.297066	-
F_{10}	Foundation Layer Alone E_F=10000 psi	2.376126	2.376041	2.290710	2.160*
$F_{10}S$	Coupled Foundation and Structure, E_F=10000 psi	2.376127	2.376041	2.290724	-
F_{26}	Foundation Layer Alone E_F=26000 psi	1.478357	1.478305	1.425214	1.341*
$F_{26}S$	Coupled Foundation and Structure, E_F=26000 psi	1.478336	1.478327	1.427747 (Structural Mode)	-
F_{29}	Foundation Layer Alone E_F=29000 psi	1.399674	1.399624	1.349359	1.267*
$F_{29}S$	Coupled Foundation and Structure, E_F=29000 psi	1.422354 (Structural Mode)	1.399656	1.399636	-
F_{50}	Foundation Layer Alone E_F=50000 psi	1.062636	1.062598	1.024437	0.969*
$F_{50}S$	Coupled Foundation and Structure, E_F=50000 psi	1.413355 (Structural Mode)	1.062623	1.062611	-
F_{100}	Foundation Layer Alone E_F=100000 psi	0.751397	0.751371	0.724386	0.693*
$F_{100}S$	Coupled Foundation and Structure, E_F=100000 psi	1.408278 (Structural Mode)	0.751389	0.751378	-

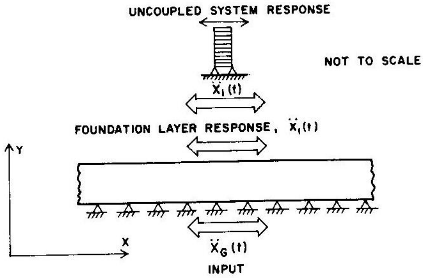
*Only shear modes considered in analytical solution. General modes considered in finite element solution.



(a) RIGID FOUNDATION SYSTEM.

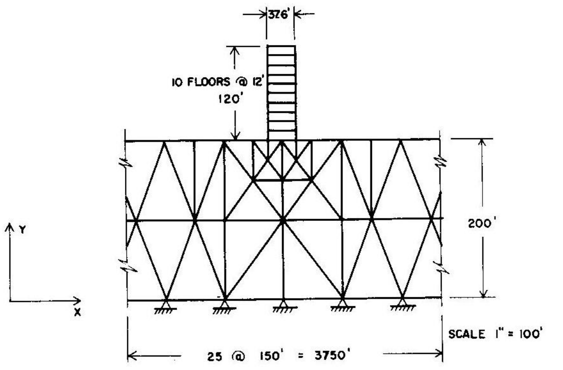


(b) COUPLED SYSTEM.

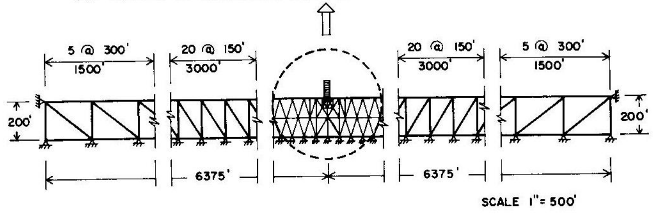


(c) UNCOUPLED SYSTEM.

FIG. 1 SYSTEMS USED IN ANALYSES.



(a) DETAILS OF STRUCTURE AND FOUNDATION ELEMENTS.



(b) FOUNDATION AND STRUCTURE: 212 NODES, 252 ELEMENTS, 256 DEGREES OF FREEDOM.

FIG. 2 FINITE ELEMENT MESH.

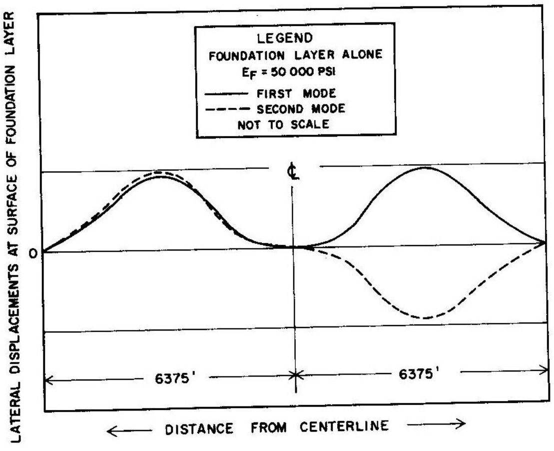


FIG. 3 FIRST AND SECOND MODE SHAPES AT SURFACE OF FOUNDATION LAYER ALONE, E_F = 50 000 PSI.

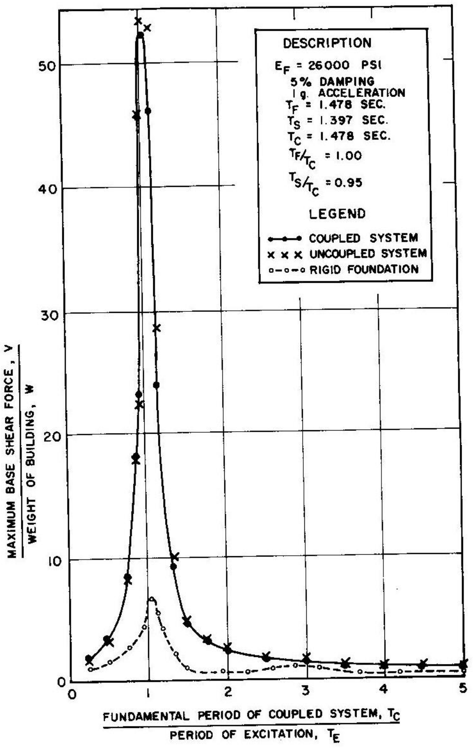


FIG. 4 RESPONSE OF THE SYSTEMS FOR E_F = 26000 PSI.

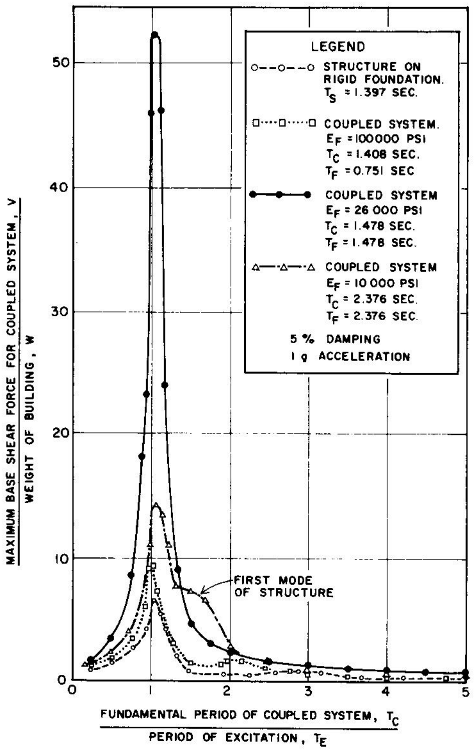


FIG. 5 RESPONSE OF VARIOUS COUPLED SYSTEMS.

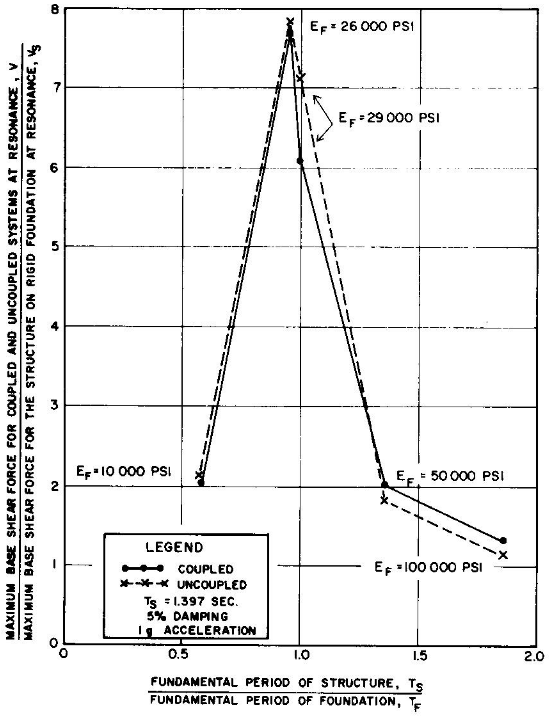


FIG. 6 RESPONSE OF THE SYSTEMS AT RESONANCE.

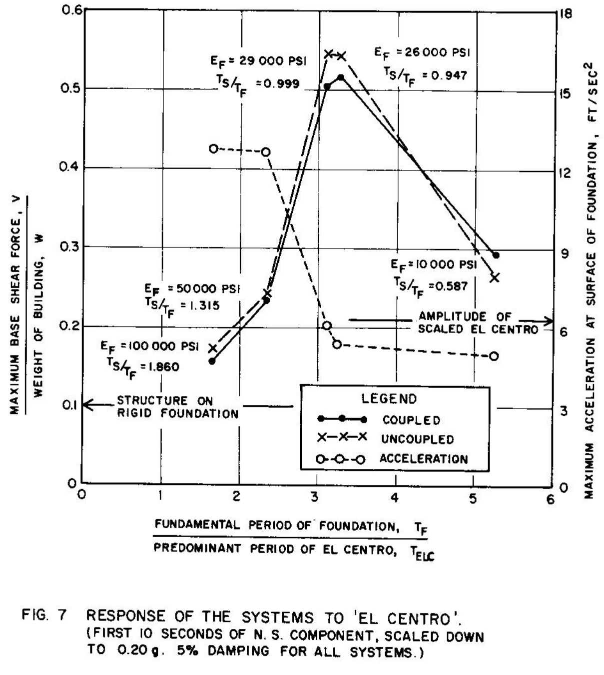


FIG. 7 RESPONSE OF THE SYSTEMS TO 'EL CENTRO'.
(FIRST 10 SECONDS OF N. S. COMPONENT, SCALED DOWN TO 0.20 g. 5% DAMPING FOR ALL SYSTEMS.)

DISCUSSION OF PAPER NO. 8

THE EFFECT OF FOUNDATION SOILS ON SEISMIC RESPONSE OF STRUCTURES

by

W.D.L. Finn, J.J. Emery and R.B. Reimer

Question by: G. McRostie

In the National Building Code (1970) simple foundation factors were retained unchanged from the 1965 Code merely to recognize that improved methods of codification were not yet available. CANCEE members are pleased that research efforts such as Paper No. 8 show promise of providing a more rational method of codifying the real variables involved in site and foundation effects.

A relationship such as the ratio of fundamental period of a building to the fundamental period of the ground at the site might be codified. Do the authors feel that they can yet suggest appropriate values for such a ratio? If so, can they suggest methods for determination of the site period since methods suitable for use in design do not appear to be available to design offices at the present time.

Reply by: J.J. Emery

The primary purpose of the paper is to indicate the amplification that may be anticipated when the fundamental periods of the structure and foundation soil are close to one another. No attempt was made in the present research to codify a relationship such as the ratio of the fundamental period of a structure to the fundamental period of the foundation soil at the site. In practice, the fundamental period for a particular structure can only be determined approximately by dynamic analysis before the actual structure is built. For the foundation soil, any dynamic analysis requires detailed information on the soil's properties that is not usually available. However, by determining the shear wave velocity, V_s, in situ and the depth of the soil layer, H, it is possible to determine the fundamental period for a uniform foundation layer from the expression for the free vibration of a semi-infinite layer with only shear deformations allowed (Equation 2).

$$T = \frac{4H}{V_s}$$

This expression gives close agreement with the fundamental period determined by a finite element dynamic analysis. For non-uniform soils, the more rigorous finite element method is required to handle the dynamic analysis.

With the approximate values of the fundamental periods of the structure and foundation soil determined, it should be possible to check for potential amplification effects. At present, the range of ratios of the fundamental periods that can lead to considerable amplification has not been examined in detail. However, as this ratio approaches unity, a complete dynamic analysis is indicated and detailed information on the foundation soil will be required.