ROLE OF FOUNDATION SOILS IN SEISMIC DAMAGE POTENTIAL

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SYNOPSIS

The seismic response of a structure depends on the stability of the foundation soils, the local ground motions and soil-structure interaction. The role of each of these factors is discussed and their relationship to building code provisions for the computation of seismic forces in structures. The report provides an understanding of the basis and limitations of existing code provisions. A thorough analysis of recommendations for including the effects of soil-structure interaction in seismic design is given.

RESUME

La réponse sismique d'une structure dépend de la stabilité des sols de fondation, des mouvements du sol localisés et de l'interaction du sol et de la structure. Le rôle de chacun de ces facteurs est discuté et leur rapport avec les dispositions du code du bâtiment pour le calcul antisismique des structures. Le rapport donne un aperçu de base et démarque les limites des dispositions du code actuel. Une analyse en profondeur des recommandations pour inclure les effets de l'interaction du sol et de la structure pour le calcul antisismique est donnée.

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INTRODUCTION

The performance of a structure during an earthquake depends on the stability of the foundation soils, the local ground motions and the interaction between structure and ground. The seismic design provisions of most building codes, of necessity, take these factors into account in a very perfunctory manner. The general free field ground motions are represented by a single parameter related to peak ground acceleration obtained from a seismic zoning map. The effect of local soil conditions are often represented by a foundation factor F, typically in the range F = 1-1.5, which increases the estimated seismic shear force in accordance with increasing flexibility of the foundation soils and soil-structure interaction is ignored.

To the extent that the zoning map represents realistically the seismic ground motions, these simple procedures have proved surprisingly effective in ensuring adequate seismic design. The failures that have occurred in buildings designed according to up-to-date code provisions may, in large part, be attributed to inadequate design of critical structural elements for anticipated dynamic loads, special or novel features of structures which resulted in behaviour not anticipated by codes and special conditions in the foundation soils which render the use of a simple foundation factor inadequate for representing the threat of the earthquake response of the foundations to the safety of the structure.

We will examine the more important mechanisms by which foundation soils affect the seismic damage potential of structures. The more important mechanisms are shown schematically in Fig. 1 which is an adaptation of a representation by Ohsaki (1). Obviously, ground failure and large differential settlements will cause severe damage to structures despite the design provisions for coping with the anticipated seismic loads in the structure itself. The potential for damage from these sources increases with increasing flexibility of the ground. The amplification of ground motions due to local soil conditions and possible resonance between the predominant periods of the ground motion and the structure increase the seismic loads on the structure and, therefore, increase the damage potential. If these factors can be foreseen and their effects are included when estimating seismic loads, then adequate resistance can be provided in the structure by proper design. The more effectively a foundation soil can dissipate energy either by radiation or material damping, the smaller the damage potential. The effects of resonance and energy dissipation are difficult to predict and are not functions of the soil properties alone. They are dependent, to an important degree, on the dynamic interaction between the structure and the soil.

In this report, we will concern ourselves in a practical way with the seismic response and stability of foundation soils and the interaction between soil and structure during an earthquake. Our objective is limited; to provide an understanding of soil behaviour during earthquakes that will help the structural engineer to understand the basis and limitations of those code provisions for seismic design that relate to ground motions and foundation behaviour and to identify the situations in which specialized geotechnical input may be necessary.

GROUND FAILURES

Ground failures may be divided crudely into 2 categories; those which cannot be predicted and those which may be assumed likely to occur because of the soil conditions at the site. The former are primarily due to sharp ground dislocations caused by random tectonic displacements or surface fracturing due to earth waves. The location of these ground fractures are impossible to predict. When they pass through a structure, the resulting warping of the structure usually results in complete loss. An example is shown in Fig. 2 which illustrates the localized violent disruption of a building due to tectonic movements generated by the earthquakes associated with the eruption of Mt. Usu near Lake Toya, Japan in 1978. There is no way in which such effects can be included in general design provisions.

One of the most significant factors leading to ground failure during earthquakes is the liquefaction of loose to medium-dense sand below the water table. Attention was focussed on this problem for the first time as a result of the widespread ground failures during the 1964 earthquake in Niigata, Japan (2). Most of the damage in Niigata attributable directly to the earthquake was associated with liquefaction and such damage has been a significant factor in most major earthquakes since then.

The mechanics of liquefaction are now well understood and the probability of occurrence can be estimated with a reasonable degree of confidence (3,4). During shaking the sand tends to compact. The water in the pores cannot escape quickly enough, at least in the finer sands, to accommodate instantaneously the compaction. Therefore, stresses are thrown on the water which increase the pore water pressure and reduce the effective or integranular stresses between the sand particles. Sand, a frictional material, depends on the effective stresses between the grains to mobilize shear strength and resistance to displacement. Therefore, the increasing pore water pressure leads to strength loss. In the extreme case, such as in Niigata, nearly all shear strength is lost and the sand behaves like a liquid with disastrous consequences for structures. Fig. 3 shows a well-designed building with little evidence of structural distress sharply tilted due to loss of bearing capacity resulting from liquefaction. In effect the building floated in the viscous fluid and assumed a position of hydrostatic equilibrium.

The sand usually liquefies first at some depth and the high pore water pressures diffuse upward. Often these high pressures vent through some local weakness in the ground, bringing up material from as deep as 30-50 feet. The vent of such an eruption is shown in Fig. 4.

When the pore water pressures are finally dissipated, usually some time after the earthquake, the compaction which was delayed by the presence of the water in the pores finally occurs and large ground settlements result. The ground settlements near a hotel due to the Niigata earthquake are shown in Fig. 5. The small box-like structure in the left foreground floated to the surface when the sand liquefied.

Another form of structural damage very typical of liquefaction is shown in Fig. 6. The bridge pier was founded on piles driven into liquefiable sands. During the earthquake, the sand liquefied and the embankments moved towards the centre of the river. The piles lost their lateral restraint in the upper regions and also deflected toward the river under the pressure of the embankment. The top end of the pier was prevented from moving by the heavy girders. The resulting lateral forces fractured the pier near the pile cap and the deformation pattern in Fig. 6, typical of many of the older bridges in Japan, resulted.

Another typical failure pattern was demonstrated during the Miyagi-ken-oki earthquake, Japan, 1978 by the behaviour of tied-back sheet-pile walls when the backfills liquefied. Under the increased pressure of the liquefied backfill material, the walls moved outward (Fig. 7). The tie-rods were attached to an anchor wall in the liquefied fill (Fig. 8) and lateral restraint was lost and large wall displacements or complete failure resulted. When the deformation of the anchor wall was limited (and hence of the retaining wall) a very typical surface failure was evident (Fig. 9). The failure pattern shows the pushing up of the ground by passive pressure in front of the anchor-wall and the subsidence behind the wall. Even heavy gravity walls can be displaced by liquefied backfill (Fig. 10).

How can soils with a high potential for liquefaction be identified and what measures can be taken to protect structures planned for construction on liquefiable sites? A wide variety of procedures are available ranging from the very complex to the crudely empirical. The more complex methods involve dynamic analysis and extensive laboratory and field testing and are justified only for the most critical structures. For most structures reliance is placed on empirical methods and judgement.

One of the earliest and simplest of empirical methods was developed by Ohsaki (5) from data gathered during the Tokachi-oki earthquake. From his investigations of sites which liquefied and did not liquefy, he concluded that the boundary between these could be described in terms of the blow counts measured during the standard penetration test. He developed the simple formula $N=2D$ in which D = depth in metres and N is the number of standard blows required to drive the penetrometer 30 cm. The application of this formula to a particular site in Niigata is shown in Fig. 11. Note that the Ohsaki line roughly describes the N values of locations at which the sand did not change in volume during the earthquake. These locations had the same N values whether measured before or after the earthquake. The areas in Fig. 11 in which the N-values increased during the earthquake are those which compacted due to shaking and hence were able to generate high pore water pressures.

There are many theoretical objections to the Ohsaki criterion and especially to its extrapolation to other sites and other earthquakes. However, it is very simple to use and should be useful for gauging liquefaction potential for situations similar to Niigata - recent alluvial deposits and hydraulic fills subjected to ground accelerations up to 0.18g and durations of shaking associated with earthquake magnitudes up to M=6.7.

A great amount of research is now being carried out to establish procedures for zoning the liquefaction potential of large-scale areas in California and in Japan. A major benefit of such studies is that they will alert developers and structural engineers to the possibility of liquefaction and the need for specialized studies of the problem in some locations. In one procedure the geological age of the deposit is used as a broad gauge for a preliminary assessment of liquefaction potential. Studies of field data show that alluvial deposits of Holocene age and hydraulic fills nearly always liquefy during major earthquakes. The incidence of liquefaction in Pleistocene deposits is much rarer and is very rare in pre-Pleistocene deposits (6). These conclusions are based on world-wide data on liquefaction during earthquakes to 1975 and have been confirmed again by the incidences of liquefaction during two major earthquakes in Japan in 1978, Miyagi-ken-oki and Off-Oshima and the Rumanian earthquake of 1977 (7).

Based on Japanese data (8), recent alluvial deposits, with the water table within a few metres of the surface, will liquefy within a radius R of an earthquake of magnitude M>6, given by the equation

$$\log_{10} R = 0.87M - 4.5$$

This equation represents the mean of the field data shown in Fig. 12, in which a lower bound is also given.

The most startling example of the effect of the age of a deposit is provided by the city of Niigata. Alluvial sand and hydraulic fills placed since the Meiji restoration of the late 19th century all liquefied. Much older sand deposits in the same city did not. The contrast is shown dramatically in Figs. 13 and 14. Fig. 13 shows the older section of Niigata after the earthquake; no damage is evident. Fig. 14 shows the newer city; the ground has liquefied and the street is under a metre of liquefied sand, with cars submerged in the liquefied material. Very recently, fundamental studies were begun in Japan to determine why older deposits behaved better during earthquakes and very interesting preliminary results have been obtained by Tohno (9).

It has been shown that the older the deposit the greater the density and hence the resistance to liquefaction. Fig. 15 shows the increase in density (and the decrease in void ratio) from late Holocene to the very old early Tertiary deposits. The very marked density increases in the pre-Pleistocene deposits indicates clearly why these do not liquefy. These increases in density would be detected by increased N-values and the resistance to liquefaction would be predicted by a criterion such as Ohsaki's.

Equally interesting is the very different kinds of contacts between grains of sand in the younger and older deposits. Finer particles tend to separate the larger sand grains in hydraulic fills and younger deposits as shown in the electron-micrographs, Fig. 16a and Fig. 16b. This leads to more unstable deposits which compact more readily under shaking. In the older Pleistocene deposits these finer particles tend to be squeezed out and there seem to be more substantial contacts between the more stable sand grains (Figs. 17a and 17b).

More recently, an empirical method for predicting liquefaction potential developed by Seed (4) has been adopted by the Applied Technology Council (10) in its recommendations for seismic design. The method is based on an estimate of the cyclic shear stresses generated during an earthquake. This procedure requires a description of the local ground motions which is much more sophisticated than that specified in the codes for estimating seismic loads. In my opinion, the procedure requires specialized skills to implement it properly and it should not be considered a routine coded provision.

What should be done if a site has a high liquefaction potential? About the only procedure that does not require specialized advice or extensive site treatment is the use of pile foundations. Pile foundations perform well provided they penetrate to a harder layer which will not liquefy and they can develop the required bearing capacity in that layer without relying on the frictional resistance of the liquefiable layer. Friction piles will not prevent large settlements or rotations if the sand in which they develop their frictional resistance liquefies. The station at Niigata (Fig. 18) provides an example of an effective pile foundation. Although the ground around the station liquefied, the pile foundations resting on firmer sands at depth showed little settlement and the station structure behaved very well. Note in Fig. 18 the substantial settlement of the surrounding ground. Other techniques for protecting the structure such as densification by vibroflotation or the installation of sand drains to control the rise in pore water pressure require high level geotechnical competence.

Loose dry sands will undergo considerable settlements due to earthquake shaking which may subject structures to damaging differential settlements. Field criteria have not been developed specifically for identifying sands which may undergo potentially damaging settlements during earthquakes. However, sands which would undergo liquefaction if saturated may be presumed susceptible to excessive settlements and the liquefaction criteria may be used to identify them. In one respect, seismically induced settlements in sands may be quite different from those under static loads; static settlements occur in the upper layers of sands whereas the major source of seismic settlements may be at some depth. Some appreciation of the nature and occurrence of seismic settlements may be gleaned from a paper by Finn and Byrne (11). The method of settlement calculation described there, however, is not suitable for routine design.

Instability problems may also be encountered in clay foundations in which sensitive clays loose their strength as a result of structural collapse due to shaking. Failures of this kind are not as widespread as liquefaction failures. No routine criteria have been established for identifying troublesome clays.

GROUND MOTIONS

Ground motions at a site generally consist of two kinds of waves, body waves which propagate up from the underlying bedrock and surface waves that are transmitted along the surface layers of the soil. The body waves are of higher frequency and are attenuated more rapidly with distance from the site. Thus at large epicentral distances the surface waves may be the more important component of the motions. In the near field, the short period body waves are most significant.

In the analysis of site response it is usually assumed that the motions are caused by shear waves propagating vertically upwards. The site is assumed to be either a linear or non-linear shear beam. In the latter case, the site is commonly analysed by an equivalent linear method developed by Schnabel et al (12) although true non-linear methods are now becoming available (13).

The results from such linear analyses have conditioned much of the present thinking about site response to earthquakes. In particular, the linear analyses led to the idea that all sites possessed a distinct fundamental period and that motion would be greatly amplified at this period. Ground motions at some sites with deep flexible surface layers, such as Mexico City (14) and Hachinohe, Japan (15) did exhibit this kind of response but very many sites showed fairly broad band response over a significant period range. In spite of the difficulties with one-dimensional shear beam analysis, in the absence of sufficient strong motion data, it was used extensively to estimate site response and to generate response spectra. With the great increase in strong motion data, especially due to the San Fernando earthquake, there is now greater reliance on field data in the predicted site response. At the same time, data on the vertical distribution of motions in the ground is being accumulated especially in Japan and this data is proving particularly useful in checking the reliability of our methods of dynamic response analysis for level sites.

A critical test of the shear beam method of analysis is described in (16). Base rock motions were measured in a borehole at a depth of 3.5km and at various elevations up to the surface and compared with computed values obtained by shear beam analysis using the measured base rock motions as input. The geological profile of the site is shown in Fig. 19a for the total depth of 3.5km and the shear wave velocities are shown for the top 100m drawn to a larger scale. A layer of soft material about 20m thick with a low shear wave velocity exists at the surface. Below this the wave velocities generally exceed 400 m/s in the top 100m.

The motions computed by shear wave analysis agree very well with the measured motions except near the surface in the soft upper layer in which the computed motions are less than those measured at the location. Decomposition of measured ground motions at the surface showed that surface waves (Love type) existed in the soft surface layer. These waves were separated from the recorded motions. When the wave was added to the motions computed by shear beam theory the computed motions compared very favourably with measured motions (Fig. 20).

It is rare indeed that the input motions at the level of base rock or of a layer substantially more rigid that the surface layers are known. In other cases, complex analyses guided by a great deal of experience and judgement are required to establish adequate base motion input for site analysis.

For most structures the trend is away from site specific analysis. Instead, suitably scaled dynamic response spectra are selected for the site and analysis is based on code provisions utilizing spectral concepts.

Seed et al (17) have developed response spectra based on recorded ground motions which are classified according to soil conditions at the site. Site conditions were arbitrarily divided into four categories, rock, stiff site conditions, deep cohesionless soils and soft to medium clays and sands. Average response spectra for these conditions are shown in Fig. 21 normalized to maximum ground acceleration. These spectra reflect the more important effects of local site conditions on ground motions. The stiffer materials show relatively narrow bands of peak amplification, the softer and more flexible sites, especially the soft clays, show a tendency to broad band response. These latter sites show substantial amplification of motions over the range of longer periods. These spectra indicate increased seismic loading for taller and more flexible structures on the softer sites.

These normalized spectra have been refined by the Applied Technology Council (10) as shown in Fig. 22 and recommended for design. The rock and stiff soil sites are lumped together, reducing the site classification to 3 types.

These spectra are based on free field motions. Any effects the structure might have on the local ground motions is ignored. In effect, the structure is assumed to be founded on a rigid base moving with the free field motions. To explore the effects of a structure on local motions, we must study the effects of soil-structure interaction.

SOIL-STRUCTURE INTERACTION

The design load provisions of building codes are based on the assumption that a structure rests on a rigid base moving with the free field motions. This assumption ignores any effects of the interaction between structure and foundations of soil. There are 3 principal effects of soil-structure interaction:

1. The free-field motions are modified under and adjacent

to the structure,

2. The flexibility of the base soil increases the flexibility of the structure and therefore the fundamental period of vibration, and

3. Effective damping of the structure is increased by the dissipation of energy by material damping in the soil and by the radiation of energy away from the structure (radiation damping).

We would like to know under what circumstances these effects are likely to be important and how they may be taken into account in a simple fashion.

Whitman (18,19) helped considerably our understanding of how soil-structure interaction affects the free-field motions by treating separately the effects of structural stiffness and mass using the concepts of <u>kinematic</u> and <u>inertial</u> interactions. These concepts are defined in Figs. 23a and 23b.

The embedded massless structure in Fig. 23a changes the local stiffness of the ground and therefore its response to the prescribed base rock motions. The process is called, appropriately, kinematic interaction, as the massless structure cannot generate any inertial effects. If the ground motions are assumed to be stationary shear waves propagating vertically then it is clear that there will be no kinematic interaction when the structure is at the surface. The magnitude of the effects of kinematic interaction depend on the stiffness of the structure and the depth of embedment.

Inertial interaction analysis takes into account the additional effects of the mass of the structure. The only disturbing forces used in this analysis are inertial forces applied to the structure as shown in Fig. 23b. The inertia force on each mass is given by the product of the mass and the absolute acceleration of the mass location as determined in a kinematic analysis. It should be noted that, when kinematic interaction effects are not significant, the free-field accelerations may be used to determine the inertial forces.

The sum of a kinematic and inertial analysis is a complete solution of the soil-structure ineraction problem. The results of such an analysis are shown in Fig. 24 for a massive deeply embedded structure. The results of an inertial analysis only are also shown. The difference between the results of the two different analyses are the effects of kinematic interaction. These are very significant for structures of the type analysed.

Complete interaction analyses are complicated and expensive and are conducted for special structures only, such as nuclear reactor structures. For most ordinary structures complete interaction analyses are neither practically nor economically justified. For such structures we need to know how important soil-structure effects are and how to take them into account simply when it is necessary or desirable to do so.

It is clear that building code provisions for determining lateral load requirements neglect kinematic interaction since they assume that the appropriate ground motions are the free-field motions. By assuming that the structures are founded on rigid bases, these provisions also neglect certain features of inertial interaction; the elongation of the natural periods and the changes in effective damping. Studies by Finn et al (20,21) have shown that for many structures covered by code requirements, it is justified to neglect kinematic interaction. The studies by Veletsos and his research group at Rice University over the last 6 years have provided the data for assessing the importance of inertial interaction. This work has been summarized by Veletsos (22) in a very comprehensive paper which provides guidance on when inertial interaction effects may be important and proposes simple methods for taking these effects into account. Veletsos' proposals have been adopted by the Applied Technology Council (10) in its "Tentative Recommendations for the Development of Seismic Regulations for Buildings".

Veletsos analysed soil-structure interaction effects for a variety of simple structures such as that in Fig. 25 on elastic and viscoelastic half-space foundations. He showed that the period of the flexibly supported structure, $\overline{T}$, is given by

$$\overline{T} = T\left[1 + \frac{k}{K_x}\left(1 + \frac{K_x h^2}{K_\theta}\right)\right]^{\frac{1}{2}}$$

in which K_x is the dynamic translation stiffness, K_θ the rocking stiffness, k the lateral stiffness of the structure on a stiff base and h, the height of the structure. The effective damping of the structure, β, is given by

$$\overline{\beta} = \overline{\beta}_o + \frac{\beta}{(\overline{T}/T)^3}$$

in which $\overline{\beta}_o$ is the foundation damping in the ground and β is the structural damping. Since $\overline{T} > T$, it is clear that soil-structure interaction reduces the effectiveness of structural damping but compensates by the inclusion of $\overline{\beta}_o$ which includes both radiation and material damping.

The relative importance of the components of foundation damping depends on the geometry of the structure as specified by the ratio of the height, h, of the structure to the equivalent radius, r, (ATC, 10) as shown in Fig. 26. For tall structures (e.g., h/r = 5), the rocking mode becomes predominant and radiation damping becomes negligible. In this case, the effective damping of the foundation soils is largely dependent on the mobilization of material damping. On the other hand, for rather squat structures, radiation damping is dominant and material damping is not very significant (Fig. 26, h/r = 1). The relative importance of radiation and material damping depends on the ratio h/r.

The effective period $\overline{T}$ and effective damping $\overline{\beta}$ characterize a single degree of freedom, fixed base system which has the same response as the original structure on its foundation. The effects of soil-structure interaction on the real structure are taken into account by the modifications of the period and damping. This modified one-degree of freedom system is called the replacement oscillator. The concept of such an oscillator was first advanced by Rainer (23). The replacement oscillator allows the use of response spectra which include the effects of soil-structure interaction. Techniques for including soil-structure interaction effects based on the concept of the replacement oscillator are included in the recommendation of ATC (10).

Various general recommendations have been suggested for deciding when inertial interaction effects may be important. Finn et al (20,21) have suggested that interaction is important when the base shear of the fixed base structure is significant in relation to the dynamic stiffness of the foundation. Veletsos (22) suggests interaction effects should be included when $\overline{T}/T > 1.08$.

We will now consider two examples of soil-structure interaction to illustrate our discussion of interaction effects.

Example 1. Hollywood Storage Building

A very interesting example of soil-structure interaction observed during the San Fernando Earthquake, 1971, has been presented by Newmark et al (24). Motions were measured in the basement at the S.W. corner of the Hollywood Storage Building and in the free-field in the P.E. lot, 112 ft. away. The building is 51 ft. in the N.S. direction, 217.5 ft in the E.W. direction and is supported on piles. Response spectra for basement and free-field motions have been reported for both the 1971 San Fernando Earthquake and the 1952 Kern County Earthquake.

Typical response spectra for the San Fernando Earthquake (35 km away) are shown in Fig. 27, for the basement and free-field motions at 5% of critical damping. It may be seen that, for periods less than 0.4s, the ordinates of the response spectrum for the basement motions are 2-2½ times less than those for the free-field. There is little difference between the spectra for longer periods. The fundamental periods for the structure in perpendicular directions are 0.5 and 1.2s. It would seem that there is little inertial interaction since at these periods the free-field and basement spectra are quite similar. It remains to explain the spectral modifications in the short period range.

If we persist in our assumption that earthquake motions consist of stationary shear waves propagating vertically, we are forced to conclude that there is little kinematic interaction either. However, if we consider that the earthquake motions are a travelling wave, then the base of the building will interfere with the free passage of the waves because of its stiffness and there will be effects of kinematic interaction. By calculating the transit time for these waves across the foundation and averaging the free-field motions with respect to the transit time, Newmark et al (24) were able to generate the response record for the basement from the free-field records. This example is important because it demonstrates that the assumption that free-field motions are stationary shear waves propagating vertically may be inadequate to explain some important observed phenomena during earthquakes.

In the case of the Kern County earthquake there was very little modification of the free-field spectrum by the building. This earthquake was located about 120 km away so that longer period surface waves would be expected to predominate at the site of the building. We would expect little kinematic interference with these waves. These results imply that interaction effects depend not only on the dynamic properties of the structure and the local foundation soils but also on the characteristics of the incoming waves.

Example 2. Pile-Supported Apartment Building

Kawamura et al (25) have been conducting a continuing study of soil-structure interaction on a 7-storey R.C. apartment building on a pile foundation. The fundamental periods of the building in the two perpendicular directions are 0.19 s and 0.24 s. The foundation soil are mainly sands but contain some silt and clay.

The building has 3-component accelerographs on the roof (RF) and the first floor (IF) and at three points in the ground below the building, GL-4m, GL-12m, and GL-24m. The ground, 15m away, was instrumented at the same three locations. The building, showing the locations of the accelerographs is illustrated in Fig. 28a.

Measurements have now been obtained during more than 80 earthquakes. Most of the earthquakes are of magnitude 6 and more than half of them less than 80km away. Ground motions at the site, however, are of generally such low intensity that they may be analysed on the basis of visco-elastic behaviour. Earthquake records at ground level within the building and in the free-field are shown in Fig. 28b. The ratio of spectral ordinates of these motions (IF/GL) are shown in Fig. 28c. Once again, we see that in the short period range the free-field spectra are reduced by about a factor of 2. In this case, there is also some amplification of the free-field motions in the long period part of the spectrum. There is, however, large amplification of the spectral ordinates in the period range 0.3-0.4. We will see later that this is the period range for the replacement oscillator for this structure. It would seem that for this structure there is some inertial interaction.

Periods and modal damping ratios for the free-field, the structure (allowing base rotation) and the coupled soil-structure system were evaluated by the spectrum fitting analysis described in Fig. 29. The results are tabulated in Tables 1, 2 and 3. These results clearly illustrate the effects of inertial interaction discussed earlier. Note that fundamental period of the first mode in the x-direction has increased from 0.237 to 0.303 s and the associated damping ratio has increased from 1.8% to 2.6%. It is important to note that the damping ratio for the coupled system is not the sum of structural and soil damping but considerably less, 2.6% compared to 9.9%.

The ratio $\overline{T}/T$ = 1.20 so that, according to Veletsos (22) and ATC (10), soil-structure interaction effects should be taken into account in the design of this structure. The significant amplification of spectral ordinates in the period range 0.3-0.4 s would seem to support this view although it does appear to be a very narrow band phenomenon.

It is clear from this field example that soil-structure interaction, as stated earlier

(1) changes the free-field motions

(2) lengthens the period of the structure

(3) changes the effective damping.

Generally, soil-structure interaction reduces the expected seismic forces in a structure and so neglecting its effects is usually conservative.

CONCLUSION

Code provisions based on good seismic zoning maps which also include a factor to reflect foundation conditions or site-dependent spectra can lead to effective seismic design provided the foundation soils remain stable.

The main causes of instability are liquefaction of saturated loose sands or excessive settlements in unsaturated sands. Preliminary screening of potentially troublesome sites can be conducted by geological rating of the age of the deposit, Holocene deposits being susceptible to liquefaction and settlements with pre-Pleistocene deposits being relatively safe. More refined screening can be carried out by using empirical criteria based on the results of the Standard Penetration Test. Sites which show a potential for liquefaction or settlement based on these tests should be investigated by a geotechnical engineer experienced in soil dynamics.

Soil-structure interaction, by increasing the effective damping for most structures, tends to reduce the seismic design loads. Its neglect therefore is conservative. However, our quantitative estimates of its effects are largely based on theory and therefore to a degree are uncertain. It may be prudent to delay introduction of soil-structure interaction provisions in the Canadian code until more field data has been accumulated and evaluated.

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TABLE 1

Modal Damping Ratios of the Building

(Base Rotation Included)

	Order	Period (s)	Earthquakes			
			No.07	No.14	No.49	No.58
X	1st	0.237	0.018	0.073	0.064	0.057
Y	1st	0.270	—	0.034	0.066	0.062
	4th	0.084	0.046	0.015	0.060	0.060

TABLE 2

Modal Damping Ratios of the Soil Column

( ):only for No.49, No.58

	Order	Period (s)	Earthquakes			
			No.07	No.14	No.49	No.58
X	1st	0.202 (0.228)	0.081	0.050	0.13	0.12
	2nd	0.090 (0.111)	0.050	0.048	0.057	0.086
	3rd	0.050 (0.062)	0.015	0.020	0.035	0.030
Y	1st	0.202 (0.228)	0.11	0.10	0.068	0.061
	2nd	0.090 (0.111)	0.030	0.055	0.044	0.042
	3rd	0.050 (0.062)	0.020	0.047	0.015	0.029

TABLE 3

Modal Damping Ratios of Coupled System

Order		Period (s)	Earthquakes	
			No.07	No.14
X	1st	0.303	0.026	0.042
	2nd	0.106	0.054	0.061
	3rd	0.071	0.050	0.033
Y	1st	0.306	0.027	0.039
	2nd	0.143	0.12	0.13
	6th	0.089	0.090	0.085

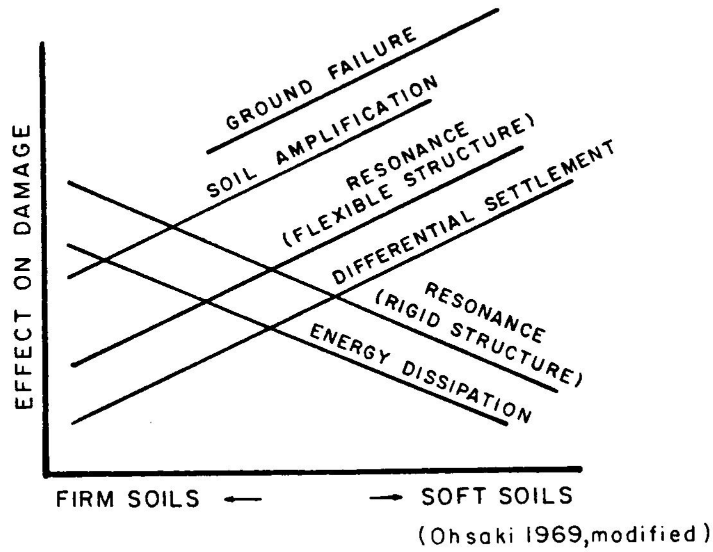


FIG. 1. EFFECTS OF FOUNDATION SOILS ON SEISMIC DAMAGE POTENTIAL OF BUILDINGS.

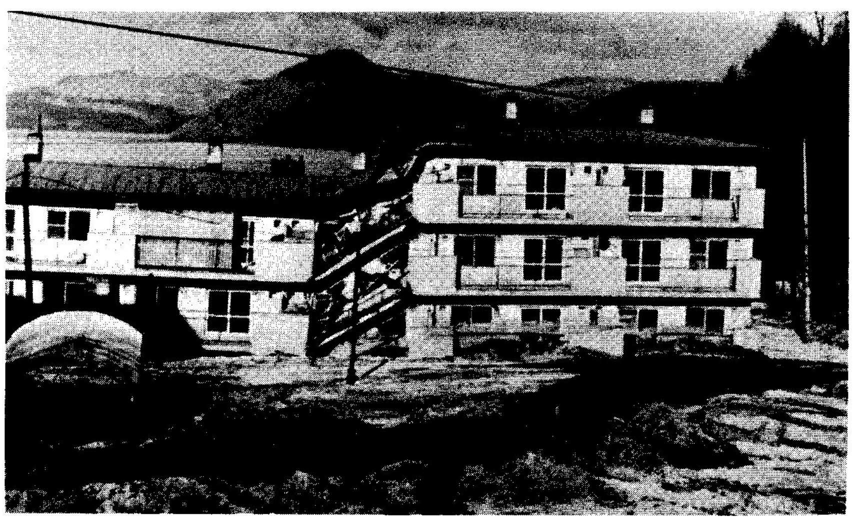


FIG. 2. BUILDING RUPTURED BY TECTONIC DISPLACEMENT, LAKE TOYA, JAPAN.

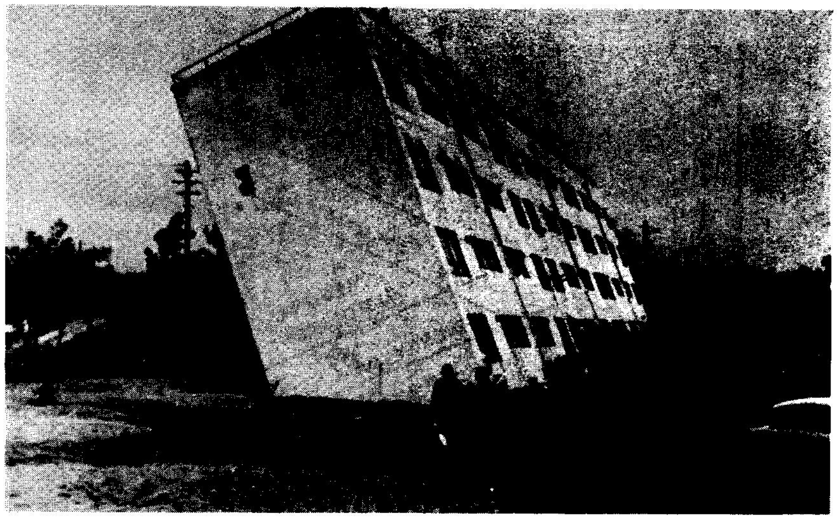


FIG. 3. TILTING OF BUILDING DUE TO SOIL LIQUEFACTION (AFTER H. KAWASUMI, 1968).

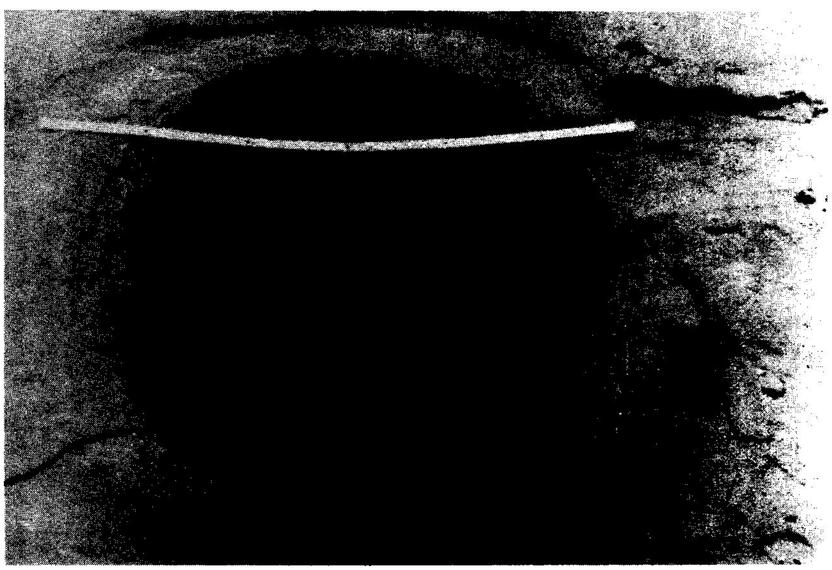


FIG. 4. VENT OF SAND BOIL.

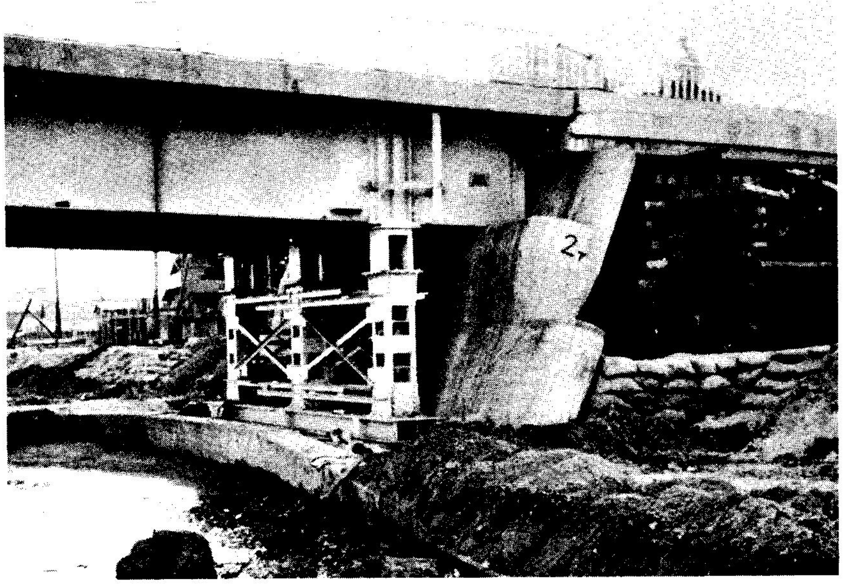


FIG. 5. GROUND SETTLEMENT DUE TO LIQUEFACTION (AFTER H. KAWASUMI, 1968).

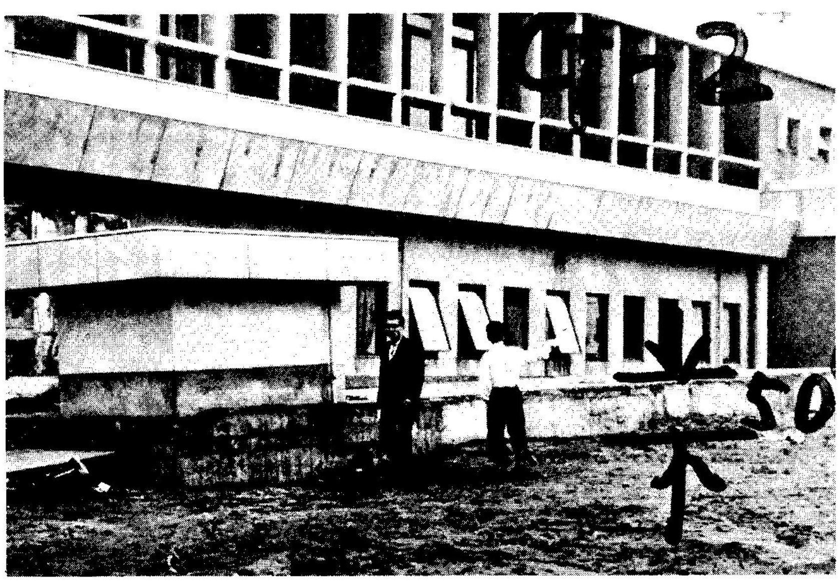


FIG. 6. TYPICAL DAMAGE TO BRIDGE PIER DUE TO EMBANKMENT SLIDING INWARDS (AFTER H. KAWASUMI, 1968).

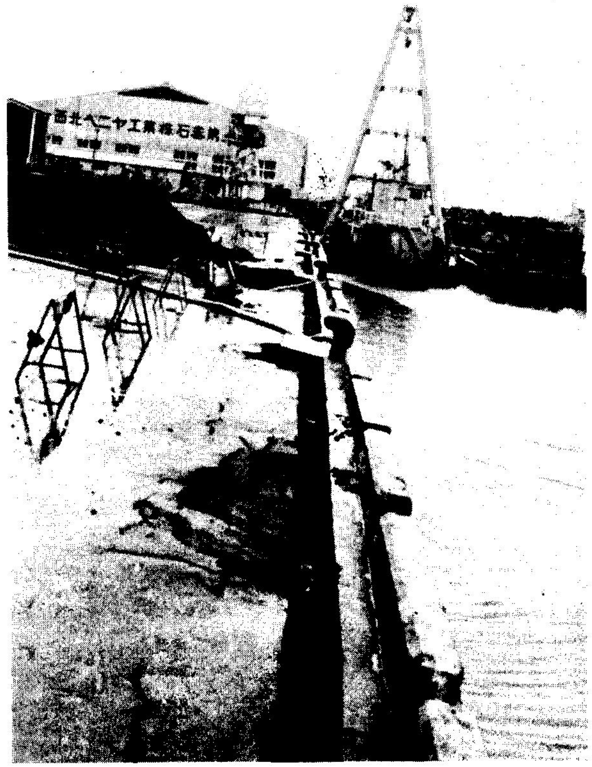


FIG. 7. DISPLACEMENT OF QUAY-WALL DUE TO LIQUEFYING BACKFILL.

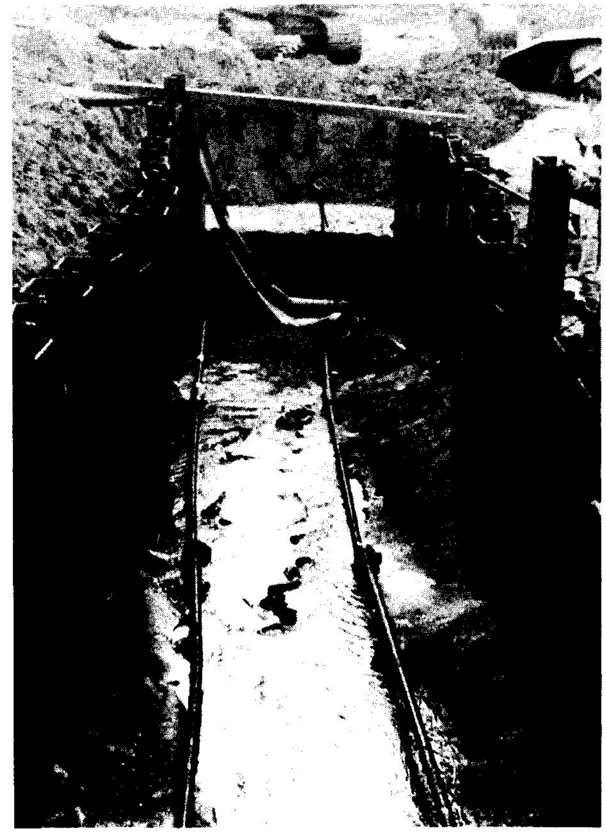


FIG. 8. TIE-RODS ATTACHED TO ANCHOR WALL.

FIG. 9. SURFACE INDICATIONS OF MOVING BURIED ANCHOR WALL.

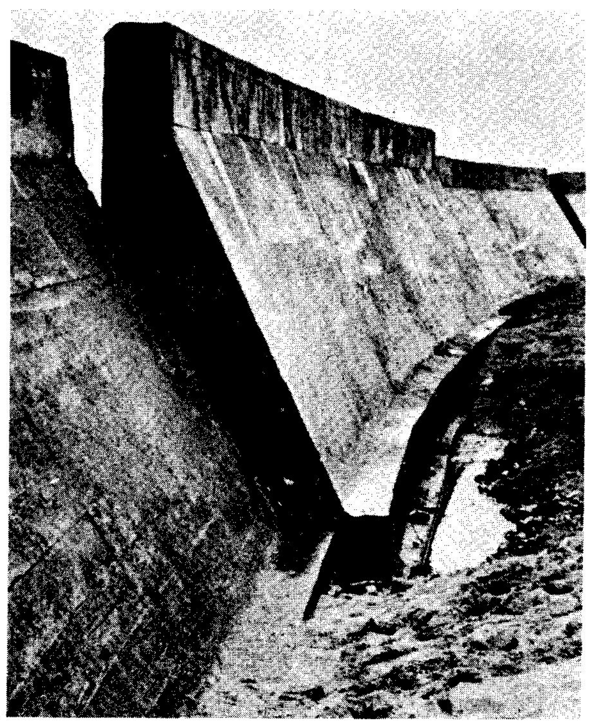


FIG. 10. GRAVITY RETAINING WALL DISPLACED BY LIQUEFIED BACKFILL (AFTER H. KAWASUMI, 1968).

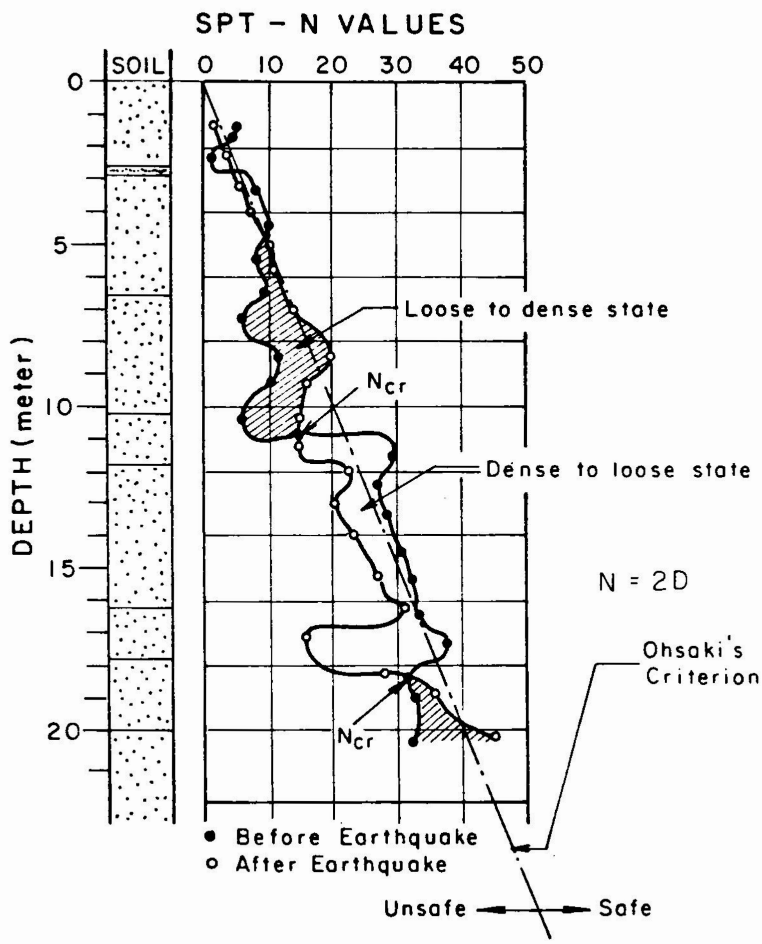
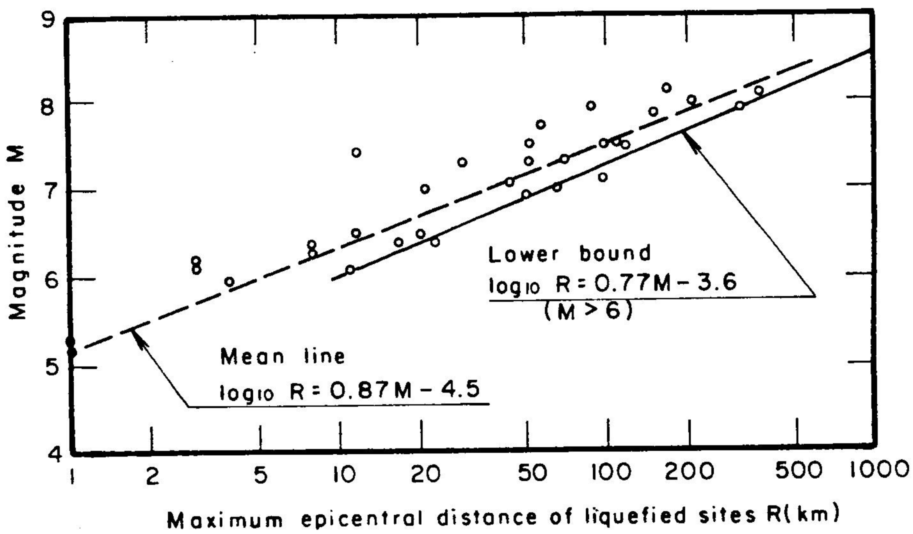


FIG. 11. VOLUME CHANGES CAUSED BY EARTHQUAKE SHAKING AND OHSAKI LIQUEFACTION CRITERION.



(After Kuribayashi and Tatsuoka, 1975).

FIG. 12. MAXIMUM EPICENTRAL DISTANCES TO LIQUEFIED SITES.

FIG. 13. OLD NIIGATA AFTER THE EARTHQUAKE - LITTLE DAMAGE ON OLDER DEPOSITS (AFTER H. KAWASUMI, 1968).

FIG. 14. NEWER NIIGATA AFTER EARTHQUAKE - GREAT DAMAGE DUE TO LIQUEFACTION OF RECENT DEPOSITS (AFTER H. KAWASUMI, 1968).

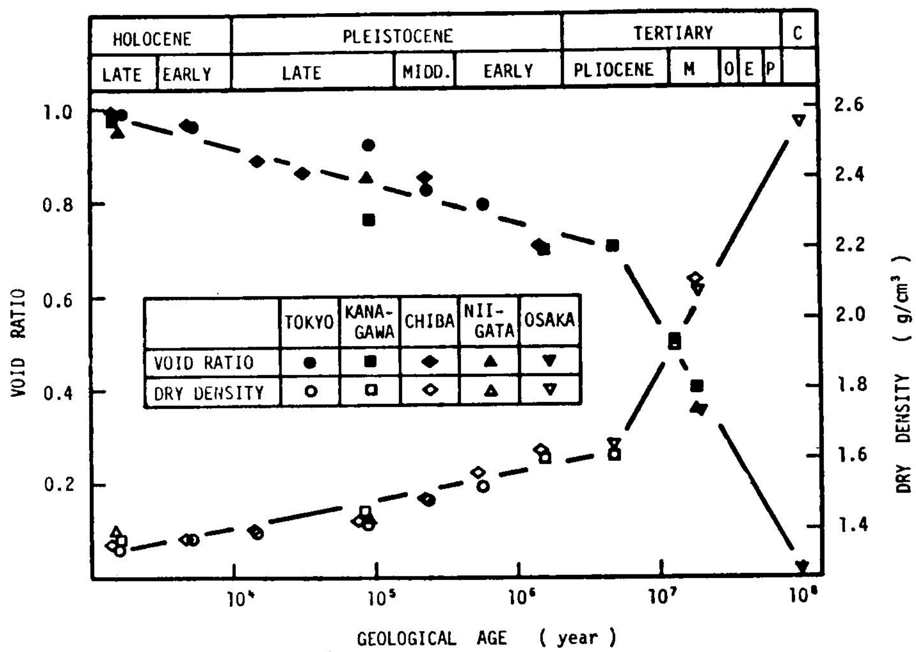
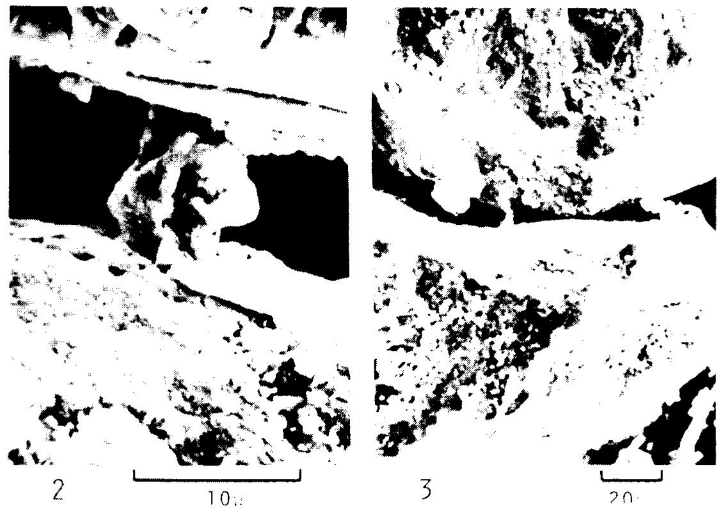
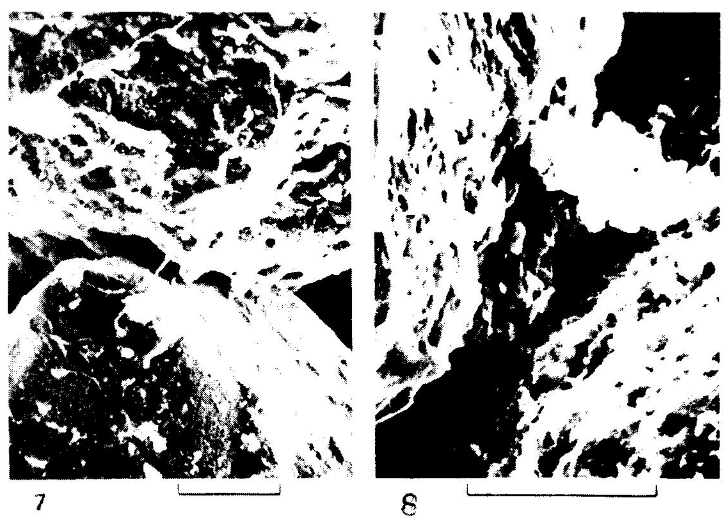


FIG. 15. VARIATION OF AVERAGE DENSITY WITH AGE OF DEPOSIT (AFTER TOHNO, 1975)



(a) (b)

FIG. 16. GRAIN CONTACTS IN HYDRAULIC FILL (a) AND HOLOCENE DEPOSIT (b).



(a) (b)

FIG. 17. GRAIN CONTACTS IN MIDDLE (a) AND EARLY PLEISTOCENE (b) DEPOSITS (AFTER TOHNO, 1975).

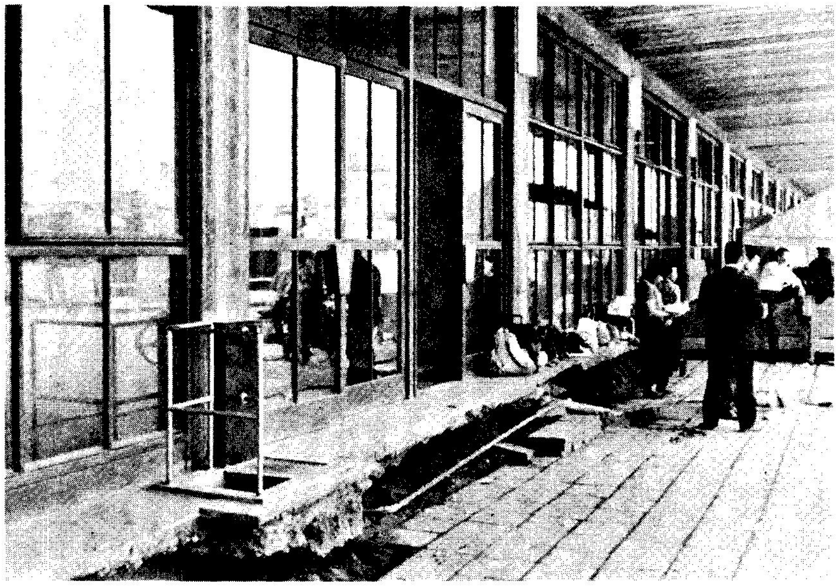


FIG. 18. NIIGATA STATION ON PILED FOUNDATION - NO STRUCTURAL DAMAGE (AFTER H. KAWASUMI, 1968).

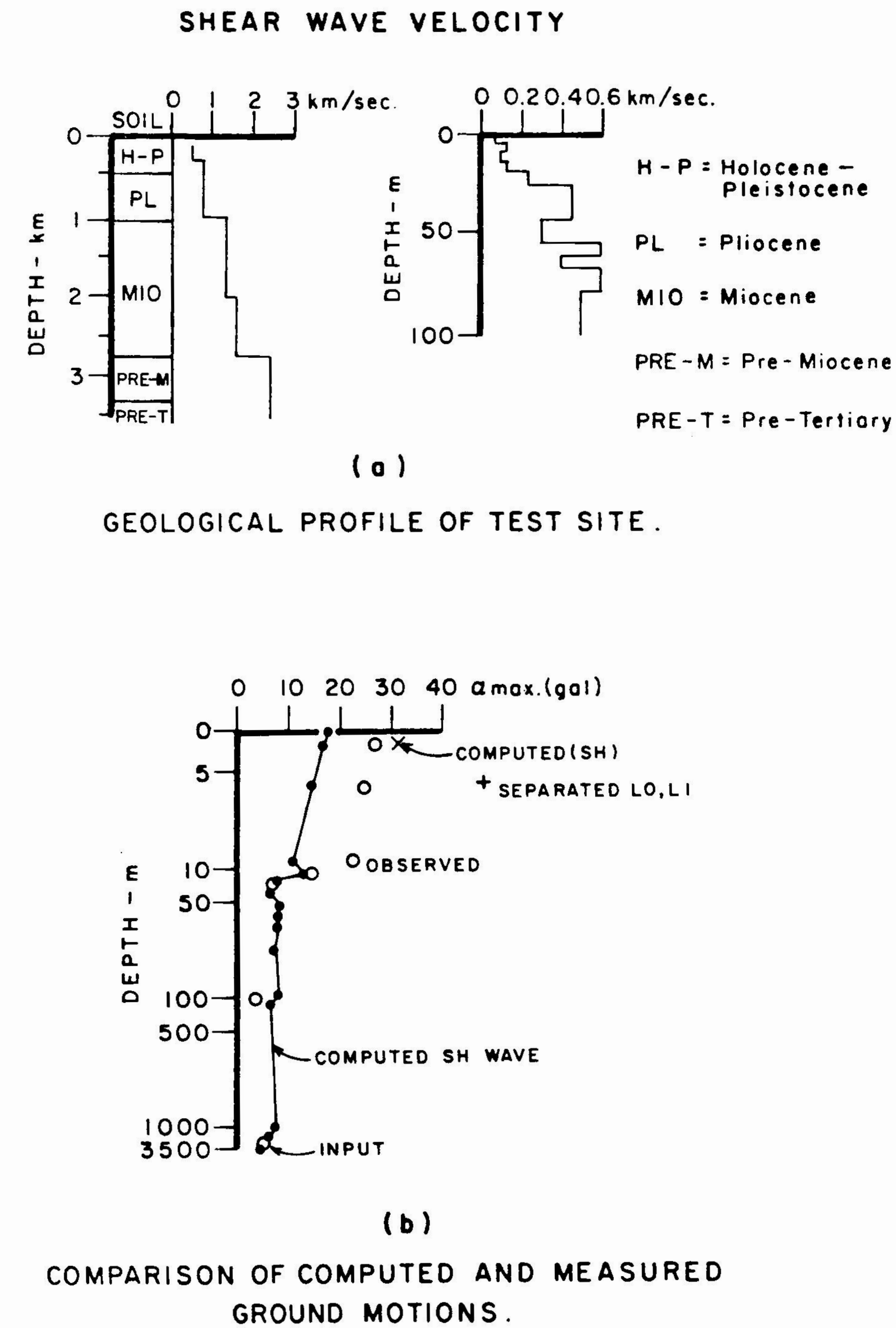


FIG. 19. FIELD TEST OF SHEAR WAVE ANALYSIS (AFTER OHTA ET AL, 1977).

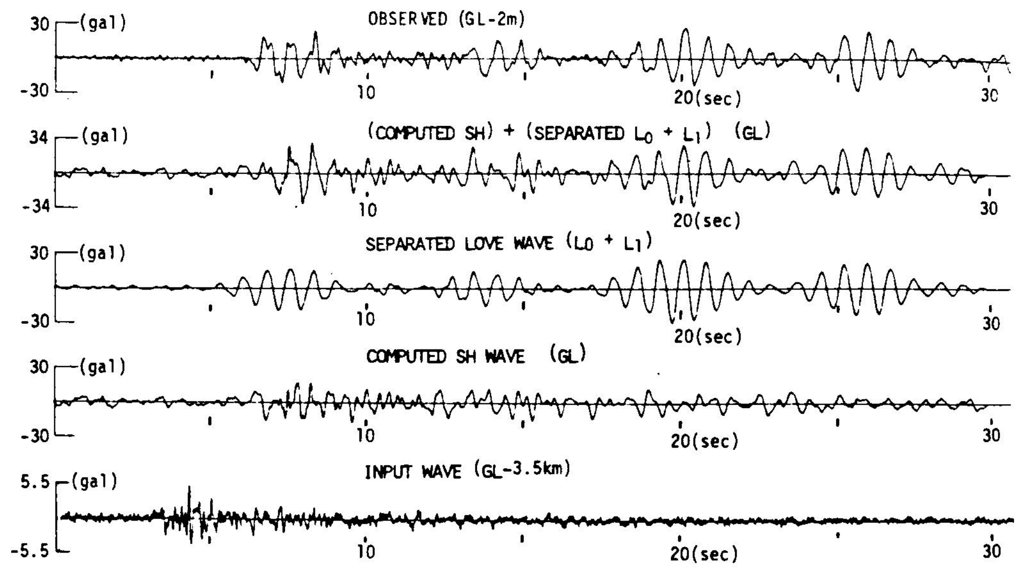


FIG. 20. COMPARISON OF MEASURED AND COMPUTED SURFACE ACCELERATION RECORDS (AFTER OHTA ET AL, 1977).

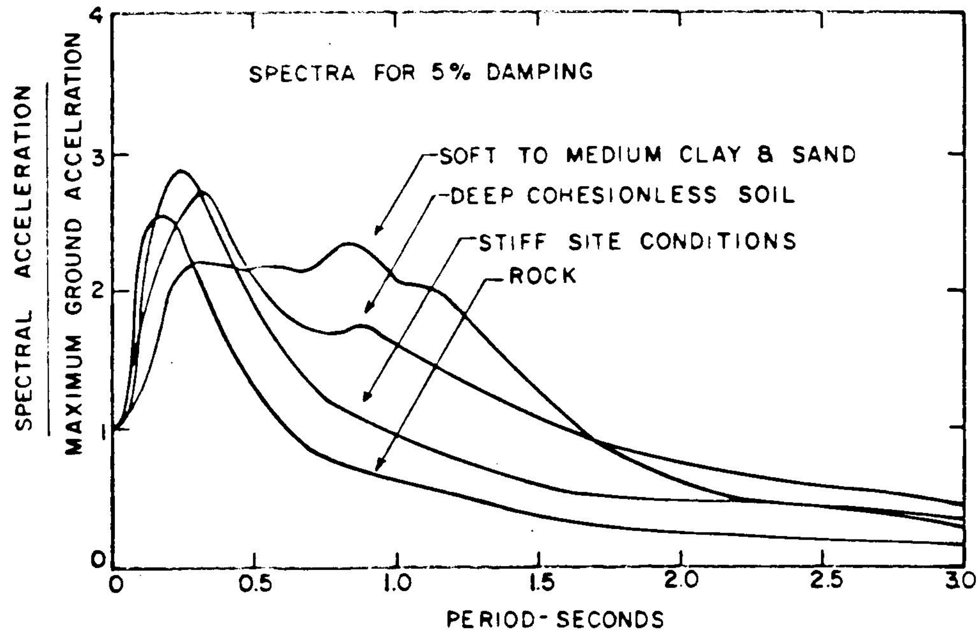


FIG. 21. AVERAGE ACCELERATION SPECTRA FOR DIFFERENT SITE CONDITIONS. (after SEED et al 1976)

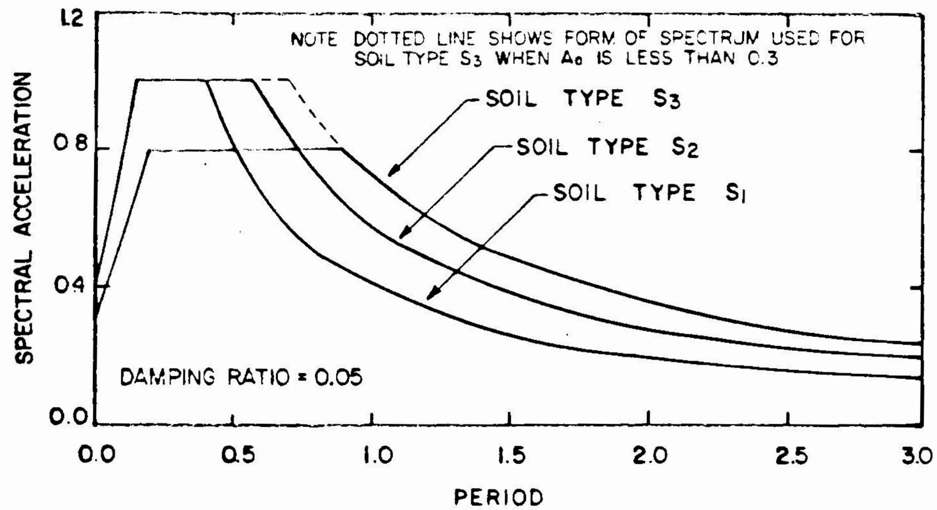
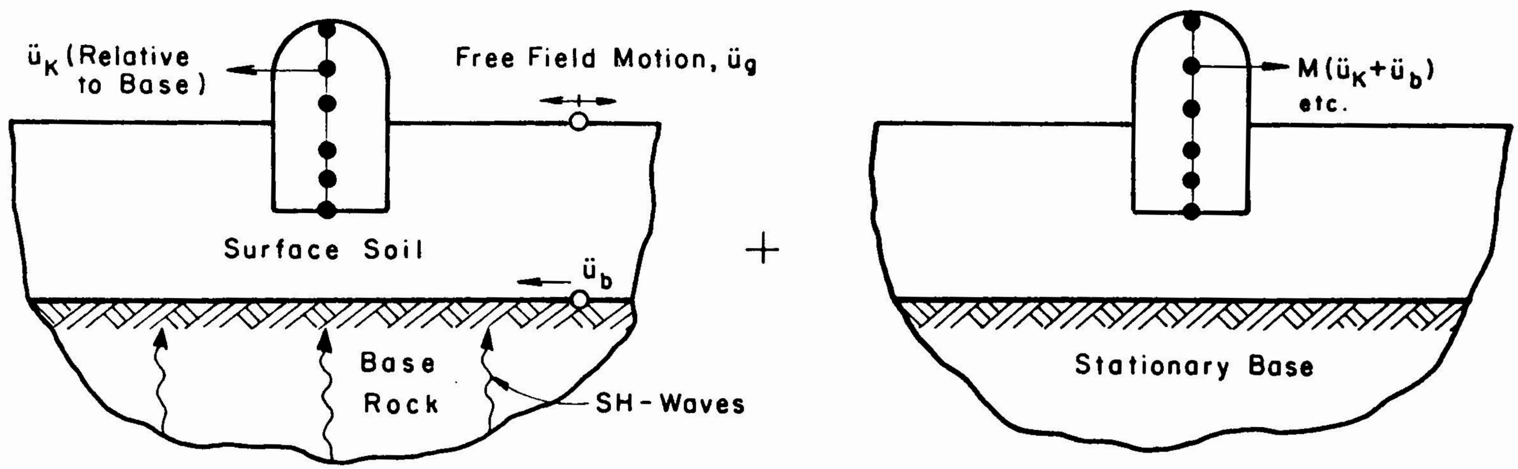


FIG. 22. GROUND MOTION SPECTRA FOR MAP AREA 7 $(A_a = 0.4)$



(a) Kinematic Interaction Analysis

Structure has stiffness but no mass.

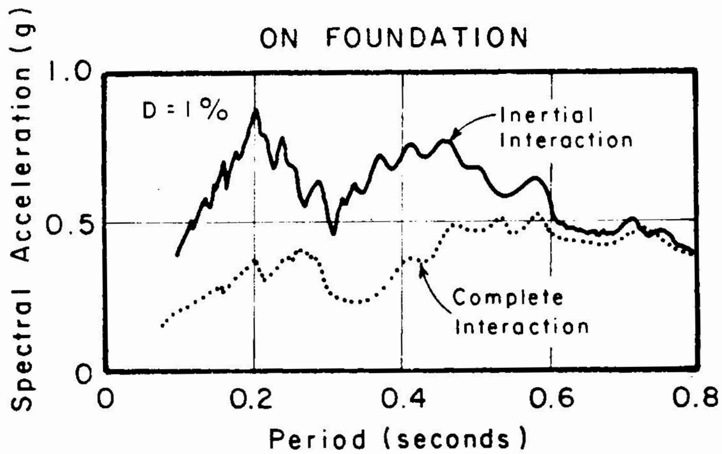
(b) Inertial Interaction Analysis

System excited by action of inertial forces active on structure alone. Motions $\ddot{u}_2$ developed relative to base.

Final Motions in structure = $\ddot{u}_1 + \ddot{u}_2$ (relative to base).

(Modified from Whitman 1976)

FIG. 23. TYPES OF SOIL-STRUCTURE INTERACTION.



(after Kausel, Stone and Webster Eng. Corp.)

FIG. 24. COMPONENTS OF INTERACTION FOR DEEP BURIED STRUCTURE.

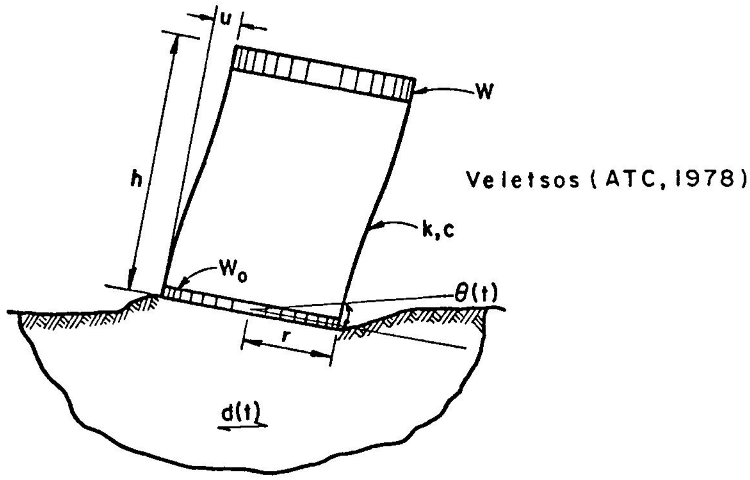


FIG. 25. MODEL STRUCTURE FOR ANALYTICAL PARAMETRIC STUDIES.

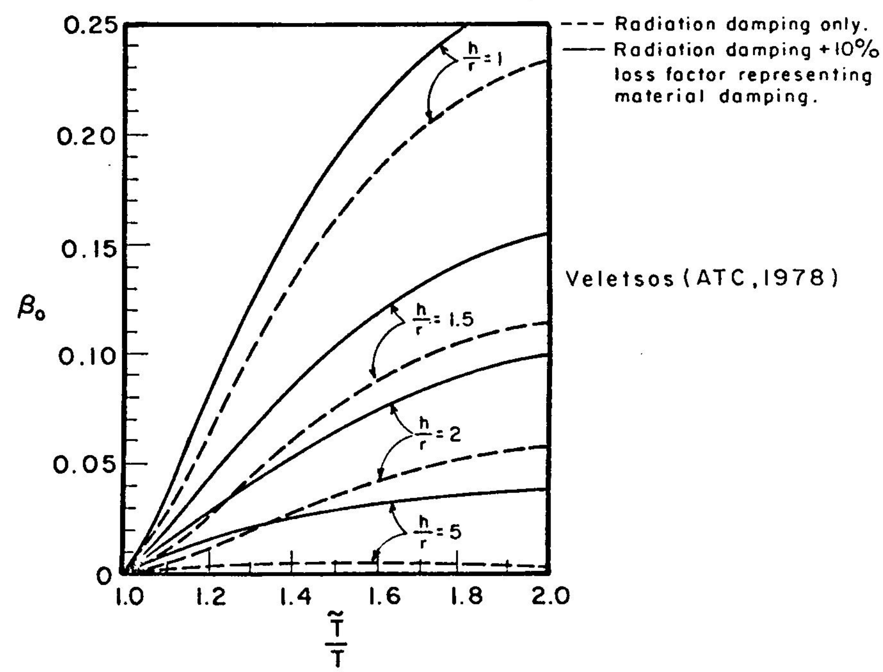


FIG. 26. EFFECT OF STRUCTURAL FORM ON MOBILIZED DAMPING RATIOS.

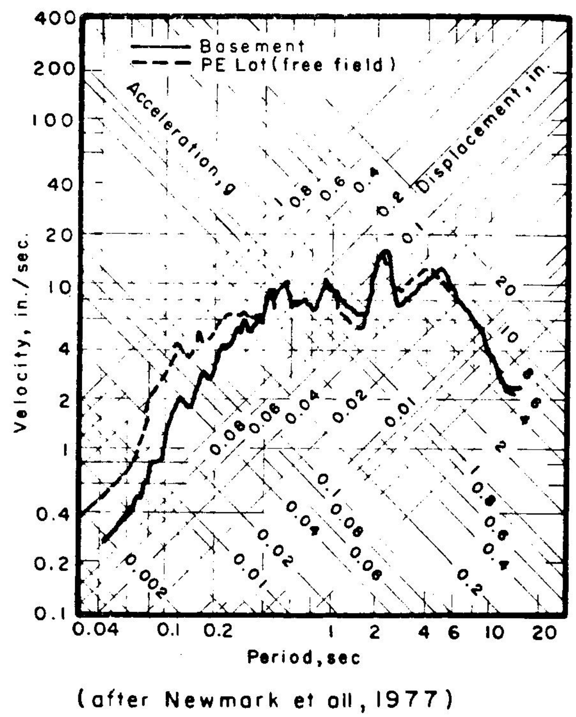
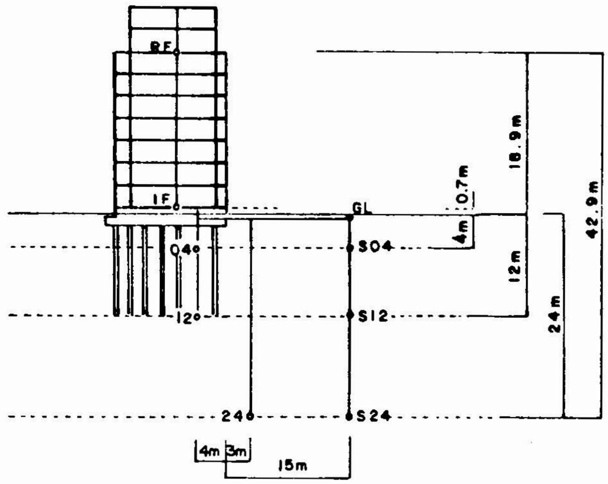
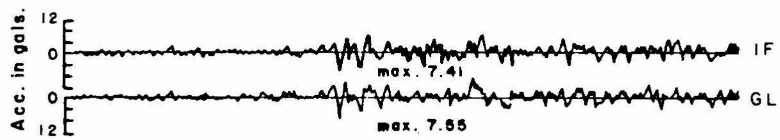


FIG. 27. EFFECT OF KINEMATIC INTERACTION ON RESPONSE SPECTRA OF RECORDED MOTIONS.



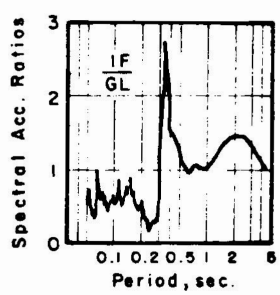
(a)

PICK-UP LOCATION AND SOIL PROFILE.



(b)

FREE-FIELD (GL) AND BASE SLAB MOTIONS (IF).



(after Kawamura et all 1977)

(c)

EFFECT OF INTERACTION ON SPECTRAL RATIOS.

FIG. 28. RECORDED SEISMIC RESPONSE OF GROUND AND BUILDING.

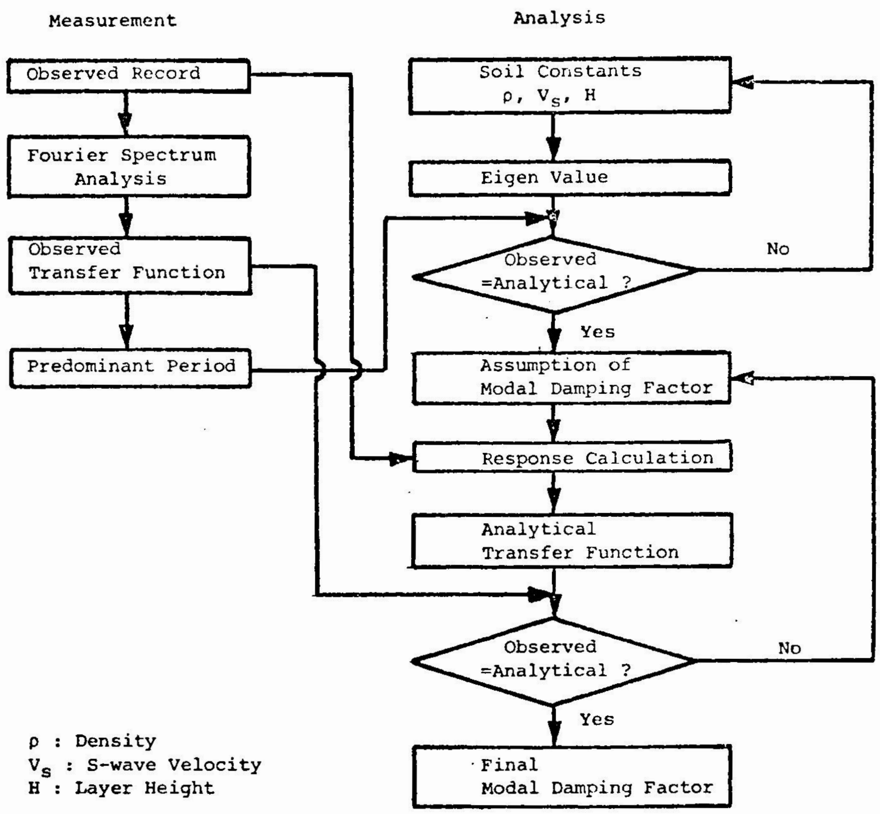


FIG. 29. PROCEDURE FOR SPECTRAL ANALYSIS OF FIELD DATA (AFTER S. KAWAMURA ET AL, 1977).