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# DYNAMIC SOIL-STRUCTURE INTERACTION IN FRAME STRUCTURES - STATE OF ART REPORT

By

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## INTRODUCTION

A comprehensive dynamic analysis of soil-structure systems is the most demanding task in realistic design of earthquake resistant structures. The two main facets of the analysis are the appropriate input parameters with suitable foundation model and the ground motion. The large computational effort and time required for the foundation analysis makes the choice of foundation model very important. Further, the uncertainties involved in defining the design ground motion representing the nature of earthquake shaking, appropriate for the site, make the problem all the more difficult.

In evaluating the response of structures to earthquakes, it is normally assumed that the motion which is experienced by the foundation of a structure is the same as the free field ground motion. This approach is strictly valid only for structures supported on rigid base. The foundation motion in such cases can not possibly be influenced by the motion of the superimposed structure and the response of the structure can be evaluated correctly by the use of free field ground motion as the foundation motion.

For structures supported on soft soils, however, the foundation motion may be influenced significantly by the motion of the superimposed structure and hence the response of the structure can be evaluated correctly only be taking into account the interaction effects between the vibrating structure, its foundation and the subsoils.

The two factors mainly responsible for the difference in the response of a rigidly or a flexibly supported structure are the variation in the degrees of freedom and dissipation of energy.

The flexibly supported structure has more degrees of freedom and therefore different response characteristics than the rigidly supported structure. The foundation of the flexibly supported structure may experience a rocking component of motion even for a purely horizontal free-field ground motion. This is certainly not possible for a rigidly supported structure. The rocking component of foundation motion results from the overturning moment induced by the lateral innertia forces and may be significant for slender and flexibly supported structures.

A substantial part of the vibrational energy of the elastically supported structure may be dissipated into the surrounding medium by rediation of waves and by hysteritic or inelastic action in the soil itself. These forms of energy dissipation are similar to structural damping which tend to reduce the response of the superstructure and are completely absent in rigidly supported structures.

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## FACTORS INFLUENCING FREE FIELD GROUND MOTION

The factors which influence the free field ground motion induced by an earthquake are :

* Distance of the site from epicentre
* Magnitude of earthquake
* Characteristics of the strata through which the waves travel to reach the site, and
* Geology and soil conditions at the site.

Ideally, the motion should be established taking into account the influence of all the factors enumerated above.

## FACTORS INFLUENCING DYNAMIC SOIL-STRUCTURE INTERACTION

The significant factors influencing the dynamic soil-structure interaction are :

* Type of structure/foundation/soil
* Degree of saturation of soil
* Innertia effect of soil
* Relative stiffness between structure and soil
* Height ratio of superstructure
* Relative mass density of structure and soil
* Relative stiffness between structure and foundation
* Adopted soil model

The effects of soil-structure interaction represent essentially the difference in the responses of the structure computed by (a) assuming the motion of its foundation to be the same as the free field ground motion and (b) considering the modified or the actual foundation motion, including the effects of energy dissipation in the supporting medium. This difference which is significant in case of massive and flexibly supported structures depends upon the characteristics of the free field ground motion and also the properties of the structure including that of the supporting soils.

## SOIL MODELS

Before reviewing the various theoretical methods incorporating dynamic interaction effects, it will be useful to briefly review the various models used to represent the soil medium. An ideal soil model should take into account -

* Soil stiffness
* Material damping
* Radiation damping
* Nonlinearity due to strain dependence, and
* Three dimensional variation of soil properties

### Equivalent Springs and Dashpots at the Base of the Structure

The most rudimentary method of modelling the soil is to use springs located at the base of the structure to represent appropriate selection of horizontal, rocking, vertical and torsional stiffness of soil (Fig. 1).

An increase in the rigorousness of the model may be affected by adding dashpots at the same location. The discrete foundation properties (spring stiffness, viscous damping etc.) for circular and rectangular footings are presented by Dowrick (1987) for all the four motions.

**Shear Beam**

This approach has been used to model soil layers overlaying the bed rock (Fig. 2), although difficulties arise in choosing appropriate stiffness and damping values for the soil. The nonlinearity of the soil may be introduced by using iterative linear analyses such as those used in soil amplification studies or by nonlinear foundation springs (Penzien, 1970).

**Elastic and Visco-elastic Half Space**

Modelling of the foundation as a homogeneous linear elastic or visco-elastic half space, in which the stiffness and damping are frequency dependent, provides very useful means of allowing for the radiation damping effect. Numerical and partly closed form formulations of the theory have been presented by Luco and Westmann (1971) and Veletsos and Wei (1971). Neglecting small coupling between horizontal and rocking motions, relationship between applied forces and resulting displacements is defined as -

$$F_j = K_j \cdot u_j \tag{1}$$

where

$j$ - denotes $x, \phi$ or $z$

$K_j$ - complex-valued stiffness (impedance) function of the form -

$$K_j = k_o \left( \beta_{kj} + i \cdot a_o \cdot \beta_{cj} \right) \tag{2}$$

where

$k_o$ - zero frequency stiffness of the foundation

$a_o$ - dimensionless frequency parameter.

Veletsos and Verbic (1973) have developed expressions for dynamic stiffness and radiation damping coefficients so as to yield solutions which compare well with the exact numerical solutions such that -

$$\beta_{kj} = 1 - \left[ \frac{b_1\, b_2^2}{(1 + b_2^2 \cdot a_o^2)} + b_3 \right] a_o^2 \tag{3}$$

and

$$\beta_{cj} = b_4 + \frac{b_1 \cdot b_2^2 \cdot a_o^2}{(1 + b_2^2 \cdot a_o^2)} \tag{4}$$

Where the parameters $b_1$ to $b_4$ are dimensionless functions of Poisson's ratio and vary for horizontal, vertical and rocking motions. In this equivalent spring-dashpot representation of the elastic half space, $\beta_k$ and $\beta_c$ are representative of dynamic stiffness of spring and radiation damping coefficient of the dashpot. Veletsos and Verbic (1973) modified the elastic parameters $\beta_{kj}$ and $\beta_{cj}$ into visco-elastic terms $\beta^v_{kj}$ and $\beta^v_{cj}$ which are given in rearranged form by Danay (1977) as -

$$\beta^v_{kj} = \beta_{kj} - a_o \cdot \beta_{cj} \left[ \frac{(1 + \tan^2 \delta)^{1/2} - 1}{2} \right]^{1/2} \tag{5}$$

and

$$\beta^v_{cj} = \beta_{cj} \left[ \frac{(1 + \tan^2 \delta)^{1/2} + 1}{2} \right]^{\frac{1}{2}} + \beta_{kj} \frac{\tan\delta}{a_o} \tag{6}$$

where

$$\tan \delta = \frac{\Delta w}{2\pi w} \tag{7}$$

Equation 7 is a representative of material damping due to hysteritic soil behaviour. $\Delta$ w represents the energy loss per cycle (area of hysteresis loop) and w, the strain energy stored in equivalent perfactly elastic material.

**Finite Element Model**

This is the most versatile method of modelling the soil-structure-foundation system. Like the half space model, it permits radiation damping and three dimensional behaviour. Also, the model has the advantage of easily allowing changes of soil stiffness both in vertical and horizontal directions. The embedment of foundation can also be conveniently handled. Although, a truly three dimensional model may work out to be expensive, yet an equivalent two dimensional model may serve the purpose in most of the cases. In order to simulate the radiation of energy through the boundaries of the element model, the following three approaches are common :

a) Elementary Boundaries - which do not absorb energy and rely on distance upto the boundary to minimise the effect of reflection of waves.

b) Viscous Boundaries - which attempt to absorb the radiating waves. The modelling of the far field is achieved by a series of dashpot and springs (Lysmer and Kulhemeyer, 1969). Accuracy of this approach is not very good for horizontal excitation.

c) Consistent Boundaries - at present are the best available absorptive boundaries which reproduce the far field in a way consistent with the finite element expansion used to model the core region. (Lysmer and Wass, 1972 and Kausel, 1974). The model by Kausel (1974) allows the lateral boundary to be placed directly at the side of the foundation with a considerable reduction in the number of degrees of freedom.

In view of the best simulation of radiation damping by the use of consistent boundaries and also in view of the earlier discussion, the finite element model gives some special advantages -

* The model has the flexibility of allowing consistent representation of stiffness and damping matrices.

* It permits radiation damping
* Nonlinearity of soil behaviour can be conveniently modelled
* Material damping can be accounted for by using a visco-elastic finite element model (Kausel and Roesset, 1975, 1976).
* The model permits time domain solution with much greater computational efficiency (Bayo and Wilson, 1983).

**Hybrid Half Space/Finite Element Model**

This method combines the advantage of the desirable factors of the semi-infinite half space and finite element methods and minimises their undesirable features. The modelling is achieved by partitioning the total soil-structure system into a near field and a far field with a hemispherical interface. The near field, which consists of the structure to be analysed and a finite region of soil around it,is modelled through finite elements. The semi-infinite far field is modelled by distributed impedance functions at the interface (Gupta, Pensien, Lin and Yeh, 1982). This model is more realistic and economical for three dimensional soil-structure interaction analyses for both surface and embedded structures.

## ANALYTICAL METHODS

Compliance of the soil foundation has been recognised as a potentially important factor in the design of earthquake resistant structures. Several methods have consequently been developed to analyse the dynamic response of flexibly supported structures. These include :

a) Model Analysis

* Parmelee (1967)
* Khanna (1969)
* Roesset, Whitman and Dobry (1973)
* Ohta, Hara, Uchiyama and Niwa (1973)
* Tsai (1974)
* Bielak (1976)
* Bielak and Palencia (1977)
* Chaku (1989)

b) Fourier Analysis

* Liu and Fagel (1971)
* Jennings and Bielak (1973)
* Chopra and Gutierrez (1974)
* Veletsos and Meek (1974)

c) Laplace Transform Technique

* Castellani (1970)
* Jennings and Bielak (1973)

d) Foss's Method

* Jennings and Bielak (1973)

e) FEM

* Isenberg and Adham (1972)
* Chu, Agarwal and Singh (1973)
* Vaish and Chopra (1974)
* Seed, Lysmer and Hwang (1975)
* Gupta, Penzien, Lin and Yeh (1982)
* Khandoker and Batra (1983)

f) Direct Step by Step Numerical Integration

* Parmelle, Perelman and Lee (1969)
* Bayo and Wilson (1984)
* Wolf (1985)
* Nogami and Chen (1985)

**Modal Analysis**

Modal superposition has perhaps been the most widely used technique in the transient analysis of time invariant linear systems. This method, however, is not rigorously applicable to the soil-structure interaction problems because the foundation stiffness and damping coefficients are frequency dependent. The building-foundation system therefore, does not possess classical normal modes. The modal superposition method used by various research workers differs in many ways, e.g. Roesset et.al. (1973) assigned weighted values of damping based on energy ratio criterion (Jacobsen, 1960) for evaluating the equivalent modal damping in composite elastic and inelastic structures whereas Tsai (1974) calculated modal damping by matching the exact and the normal mode solutions of the amplitude transfer function for a certain structural location, Bielak (1976) on the other hand, considered overall damping for a given mode of building-foundation system as a composite value made up of energy dissipated by the structure and the energy losses from internal friction and wave radiation into the foundation medium.

**Fourier Analysis**

The foundation stiffness and damping terms relating forces and displacements for a rigid foundation on a linear elastic half space depend on the frequency of excitation. The governing interaction equations of motion of the structure-foundation system are therefore written in Fourier transformed frequency domain or in the Laplace transformed domain.

The steady state response to harmonic ground motion at a particular excitation frequency is determined by solving the frequency domain equations. Response to an arbitrary ground motion is then determined by Fourier transforming the ground motion, determining the steady state response for a large range of frequencies over which the ground motion and structural response has significant components. Fourier synthesis of the frequency response is then performed in the time domain. In practice, these responses are generally performed by Discrete Fourier Transform Techniques, using Fast Fourier Transform Algorithm. This typically involves determining steady state responses for a large number of excitation frequencies. For each frequency, this entails solution of as many algebraic equations as the number of degrees of freedom for the structure including those at the structure-foundation interface. This method requires large computational effort for structures like multistorey buildings.

The more important and the fundamental drawback in the method is that it takes the advantage of the important feature that structural response to earthquake ground motion is essentially obtained in the first few modes of vibration. Standard modal analysis is not applicable because building does not possess classical normal modes when foundation interaction is considered. This is due to the dependence of the foundation properties on excitation frequency. Even if the foundation stiffness and damping are approximated by independent values, damping in the structure and foundation will not usually be so related as to permit the classical normal modes.

**Finite Element Analysis**

The analysis of the earthquake response of the soil-foundation system, idealised as an assemblage of finite elements, is best carried out by using a substructure approach in which the foundation is first analysed independent of the structure to obtain its dynamic compliance characteristics and its effect is then incorporated into the structural equation of motion. This sub-structure approach allows the response of the structure to be evaluated with a higher degree of refinement and with far greater computational efficiency. Further, it is a well-known fact that structural response to earthquake ground motion is primarily due to first few modes of vibration of the structure. Considered as a subsystem, it does not possess classical normal modes of vibration. Structural behaviour can therefore be effectively expressed in terms of very few Ritz shapes consisting of the shapes of the lower modes of an associated system. In the Ritz coordinate system, although the equations of motion do not uncouple as in the classical modal analysis, the first few equations suffice to give a good approximation to the response leading to large computational savings.

**Time-Domain Analysis**

Nonlinearity of soil behaviour can be modelled with non-linear finite elements, but the necessary time-obtain analysis is very expensive. Alternatively, nonlinearity could theoretically be simulated in repetitive linear model analyses with adjustment of modulus and damping in each cycle as a function of strain level. In frequency-domain solutions (for example, when using consistent boundaries), nonlinearity can be approximately simulated again using an iterative approach. A recent major development by Bayo and Wilson (1984) permits a time domain solution with much greater computational efficiency than was earlier possible. This is due to the use of Ritz vectors rather than exact eigen values for free vibration mode shapes. Factors that may be incorporated include structural embedment, arbitrary soil profile, flexibility of foundations, spatial variations in free field motions, interaction between two or more structures and nonlinearity of both soil and structure.

## LABORATORY STUDIES

Seismic behaviour of structure-foundation-soil system can be effectively studied in the laboratory making use of shaking table provided an adequate modelling of the system can be achieved for proper representation of the prototype. A case study of an eleven storeyed building supported on cast-in-place piles is an interesting example of the laboratory study (Mizuno et.al., 1985). In the building, earthquake observation was performed and its base fixed fundamental natural period was evaluated as 0.46 sec from the Fourier spectral ratios of the observed earthquake wave forms.

The predominant period of the subsoil was 0.71 sec and about 1.5 times as long as the base fixed natural period of the building.

**Modelling**

In modelling of the building in accordance with the similitude ratios (Table 1), one of the interior dwelling units was extracted through the height of the building and treated as a single degree of freedom system with about 1 percent damping ratio adjusted. The height of the building was ignored in modelling. Model building, an embedded base and piles were made of steel. The model piles were plates with round edges having length - 71.7 cm, width 5 cm and thickness 0.57 cm. Pile head was fixed and its tip was hinged. The ground was taken to be excited in transverse direction.

**TABLE 1**
**SIMILITUDE RATIOS**

| Items | Dimesion | Similitude Ratio |
|---|---|---|
| Length | $L$ | 1/30 |
| Weight | $MLT^{-2}$ | 1/36000 |
| Time | $T$ | $1/\sqrt{30}$ |
| Bending rigidity | $ML^3T^{-2}$ | $1/324 \times 10^5$ |
| Density of ground | $ML^{-3}$ | 3/4 |
| Velocity | $LT^{-1}$ | $1/\sqrt{30}$ |
| Acceleration | $LT^{-2}$ | 1 |

Model soil was composed of polyacrylamide and bentonite. The material was chosen due to its elastic behaviour. The prototype subsoil was approximated by two layered model. Shear wave velocities of the upper and lower model layers were 17.3 m/s and 43.9 m/s respectively. Table 2 summarizes the material and geometric characteristics of the model as compared to the prototype. This model, called as the 'basic model', represents the case where the base fixed natural frequency of the building, $f_b$, is higher than the fundamental natural frequency of the subsoil, $f_g$. Two other building types, $f_b \simeq f_g$ and $f_b < f_g$ were also selected. Table 3 gives the dynamic characteristics of the three model buildings. Figure 3 illustrates the experimental facilities and the instrumentation used for the basic model. Blocks of water saturated urethane foam were set around the cylinder-shaped soil model in order to simulate the infinite condition of subsoil and the radiation condition of waves propagating from the foundation. Preliminary tests of the apparatus confirmed good simulation. Two trenches were excavated in both lateral sides of the embedment to remove friction between the embedment and model soil. This treatment was made to preserve the condition of the prototype in the model.

Experiments were carried out to study the influence of the following parameters :

**TABLE 2**

**DEGREE OF CONSIDERATION IN MODELLING OF VARIOUS PARAMETERS (after Mizyno, 1978)**

| ITEMS | PARAMETER | DIMENSION | DEGREE OF CONSIDERATION IN MODELLING (suffix m and p denote model and prototype) |
|---|---|---|---|
| GROUND | Density | $ML^{-3}$ | ● Model Soil Density $\rho m$=1.2t/m³. Clay Density $\rho p$=1.6t/m³. Then $ML^{-3}$=3/4 is fixed. |
| | Shear Modulus | $ML^{-1}T^{-2}$ | ○ Satisfied |
| | Shear Wave Velocity | $LT^{-1}$ | ● Satisfied |
| | Depth of Layer | L | ● Satisfied |
| | Predominant Period | T | ○ Satisfied |
| | Damping Ratio of Soil | 1 | ○ Material Damping, hm 6-9%. If strain level of clay is assumed $3\times10^{-4}$-$1\times10^{-3}$, hp 5-7% |
| | Poisson's Ratio | 1 | △ Model Poisson's Ratio $\nu m$=0.22~0.39. Clay $\nu p$=0.5 |
| | Coefficient of Horizontal Soil Reaction | $ML^{-2}T^{-2}$ | ○ This parameter is function of shear modulus and Poisson's ratio. Because G is based on the similitude ratio, this parameter varies according to the variation of Poisson's ratio. |
| EARTHQUAKE | Acceleration | $MLT^{-2}$ | ○ Depending on the ability of the table. Then relation between ML and T is fixed. |
| PILE | Bending Rigidity | $ML^{3}T^{-2}$ | ● Possible by substiting steel for reinforced concrete. |
| | Diameter | L | ● Perfectly satisfied |
| | Length | L | ● Perfectly satisfied |
| | Line Density | $ML^{-1}$ | △ Ignored but the realized density was 62% of the required. |
| | Shape of Cross Section | | × Ignored. Rectangle with round edges was substituted for circle. |
| EMBEDMENT | Length | L | ● Perfectly satisfied |
| | Mass | M | ● Embedment was considered as one mass. Then mass ratio of embedment to soil was the same as the prototype. |
| BUILDING | Mass | M | ● Effective mass of fundamental mode (base-fixed) was considered. |
| | Fundamental Period | T | ● Adjusted by column stiffness. |
| | Damping | 1 | ○ Damping ratio of building was estimated as 1% and adjusted. |
| | Height | L | × Ignored |
| | Displacement | | |
| | Deformation | L | ○ Satisfied |
| | Sway Effect | L | ○ Satisfied |
| | Rocking Effect | L | × Height is ignored. So this parameter is not exactly quantitative. |
| SYMBOL | ●: Perfectly Satisfied<br>×: Ignored | | ○: Satisfied  △: Not Exactly Quantitative |

**TABLE 3**
**DYNAMIC CHARACTERISTICS OF BUILDINGS**

| Kind of Bldgs | Relation Between $f_b$ and $f_g$ ($f_b$ : Base Fixed Nat. Freq. of Bldg; $f_g$ : Predominant Freq. of Ground) | Dimensions of Building | | Base-Fixed Characteristics of Building | |
|---|---|---|---|---|---|
| | | Weight (kg) | Height (cm) | Freq. (Hz) | Damping (%) |
| 1 | $f_b > f_g$ | 12.7 | 24.8 | 11.8 | 1.06 |
| 2 | $f_b \approx f_g$ | 12.1 | 31.5 | 6.80 | 0.99 |
| 3 | $f_b < f_g$ | 20.2 | 40.8 | 3.59 | 0.98 |

* type of building
* èmbedment of the base
* type of foundation and
* pile head condition

The tests conducted were :

* static pullout tests
* free vibration tests of building set-up and
* shake table tests

The shake table tests consisted of :

* steady state vibration tests
* earthquake motion tests

Records of Off Miyagi Perfecture Earthquake (Sept. 25, 1980) observed at the pile tip of the prototype were adopted. In order to verify the effectiveness of modelling, the experimental results were compared with the earthquake observation results of the prototype.

Figure 4 shows the Fourier spectral ratios (building/pile tip) observed from the prototype and model tests. These results indicate that the modelling was appropriate.

**Significant Observations**

The results of the steady state vibration tests and the earthquake motion tests are discussed in detail by Mizuno et.al. (1985). However, the main features brought out from the study are :

* The model experiment (Mizuno, 1985) is a powerful new approach for the study of dynamic soil-structure interaction.

* The behaviour of piles during earthquake is governed not only by the mode in which the building is predominantly stimulated, but also by the mode in which the subsoil is predominantly stimulated.

* Effect of soil movement on piles during earthquake should be taken into account into the design of lateral resistance of piles, especially

in case the frequency, $f^*_b$ (where the building is mainly excited in a building-pile-soil system) is higher than the frequency, $f^*_g$ where the subsoil is mainly stimulated in the same system.

* The participation factor of piles, which is the sum of the innertia forces of the building and the base, is valid for only the mode in which the building is stimulated and the subsoil keeps still.

* Soil-structure interaction should be included in the design of piles, because the deformations of the soil may cause large bending moment in the pile.

## FIELD STUDIES

Field investigations for studying the dynamic soil-structure interaction in pile supported structures have been carried out by Kawamura et.al. (1977), Sugimura (1977) and Urao et.al. (1989). The nature of these field investigations and the information which can be obtained are illustrated here taking reference to the work by Kawamura et.al. (1977). A brief review is presented in the following section.

Kawamura et.al. (1977) conducted measurements on a seven storeyed building made of precast light weight concrete (Figs. 4a). The building had no basement floor and was supported on precast piles driven into a dense sandy layer 12 m below the ground surface. The structure components were walled frames in longitudinal direction and shear walls in transverse direction. The natural periods of the building obtained from forced vibration tests were 0.24 sec in longitudinal and 0.19 sec in transverse directions.

The surrounding soil consisted mainly of sand and partly of silt or clay. The N-value variation is shown in Figs. 4b and c, both for the land side and the sea side. The thickness of the reclaimed soil layer on sea side is as shallow as 4-5 m. The predominant periods observed in microtremor at the ground level were 0.2-0.4 sec, 0.7 sec and 1.2 sec in order of the peak height.

### Measurement System

The locations of pick-ups, shown in Fig. 4c, consisted of two groups, namely building line and the soil line. Building line consisted of 5 points - RF, IF in the building, GL -4m, -12m and -24m just below the building, Soil line was parallel to the building line and consisted of 4 points at the same level i.e. GL, -4m, -12m and -24m, each about 12 m away from the building. Every point has three components, two in the horizontal (x,y) and one in the vertical (z) direction.

### Observations

During the period 1971-77 when the measurements were taken, about 80 earthquakes of small or of intermediate intensity have been recorded. The distances to the epicentres were distributed over a wide range but more than half of them were less than 80 km. Most of the focal depths were around 50 km and majority of them were less than 100 km. The magnitudes were mostly below 6.0 with a few exceptions.

Table 4 gives the amplification factor of horizontal motion obtained on the basis of observed earthquake records. It shows that in the horizontal direction, the amplification was remarkable. The amplification factors of horizontal motion from GL-24 m to RF were 6.64 in longitudinal and 6.19 in transverse directions. In the soil line, from -24m to GL, these were 3.24 and 3.61 respectively. Vertical amplification was not so large as the horizontal one.

To know the period characteristics of the observed records, Fourier analysis was carried out and the transfer functions of the building, soil and interaction system were obtained by the spectral ratios. Similarly, the modal damping factors of the building, soil and the coupled system were calculated applying the spectral fitting method.

**TABLE 4**
**AMPLIFICATION FACTORS FROM GL-24 m**

| | | X | Y | Z |
|---|---|---|---|---|
| Building Line | RF | 6.19 | 6.64 | 2.43 |
| | 1F | 2.45 | 2.34 | 1.64 |
| | 04 | 1.32 | 1.70 | 1.55 |
| | 12 | 1.09 | 1.22 | 1.16 |
| | 24 | 1.00 | 1.00 | 1.00 |
| Soil Line | S00 | 3.61 | 3.24 | 1.96 |
| | S04 | 1.68 | 1.72 | 2.20 |
| | S12 | 1.20 | 1.05 | 1.54 |
| | S24 | 1.00 | 1.00 | 1.00 |

## DISCUSSION

Dynamic soil-structure interaction studies can be carried out either by analytical means or by the laboratory model studies or by conducting the field studies.

Various analytical methods listed earlier have their own limitations. However, the finite element analysis, particularly in time domain, seems to be the one approach which can take into account soil non-linearity and the dependence of stiffness and demping in each cycle on strain level. This approach therefore holds significant promise.

In case of the field studies, the whole operation of instrumentation has to be very carefully planned much before the start of work. The type and nature of the instrumentation has to be well identified so that the same could be carefully installed during the various stages of construction. Moreover, such field investigations can yield fruitful results in areas where the frequency of occurrence of earthquake is likely to be large, e.g. Japan. In such a case, the earthquake response of the building can be recorded every time there is a tremor. Subsequently, the data could be analysed to yield information about various aspects of the behaviour.

For laboratory studies, the shake table facility holds promise. Such a facility, if available, can best be utilised with representative model of the whole system utilising suitable material. In this respect, the work by Mizuno (1985) is unique and interesting. The modelling of various parameters (Table 2) can be taken as a good guide.

**SOME OBSERVATIONS AND POSSIBLE FUTURE TREND**

Soil-structure interaction phenomenon in pile supported structures as reviewed in the preceding sections is an interesting and complex phenomenon. Attempts have been made to solve the problem analytically developing solutions based on certain simplifying assumptions. The use of experimental techniques in the laboratory on small scale models and instrumentation of prototype structures in countries like Japan holds good promise. However, a lot more efforts have to go in to obtain a complete solution to the problem. On the basis of the status as of today, the following observations and also projections for future studies are suggested.

a) For projects in which soil-structure interaction effects are likely to be important, the choice of the analytical method requires careful consideration.

b) It should be noted that where the dynamic behaviour is expressed in frequency dependent terms, the problem must be analysed in frequency domain and not in time domain.

c) Ideally the earthquake motion should be applied at bed rock to the complete soil structure system. This is not a realistic method as, at present, not much is known about the bed rock motion than the surface motion and there is a scatter in possible results for the soil amplification and attenuation. The soil amplification and attenuation is also influenced by the presence of the structure because the effect of soil structure interaction is to produce a difference between the motion at the base of the structure and the free field motion which would have occurred at the same point in the absence of the structure. In practice, this refinement is seldom taken into account.

d) Damping in soil in different modes of vibration varies considerably. As most currently available dynamic analysis programs are written for equal damping in all modes, some intermediate value of damping has to be chosen which may lead to realistic results. The value of damping used should not vary too much from that of the mode in which most of the vibration work is done. Hence, a trial mode shape analysis may have to a done to determine which modes predominate. Use of too high or too low value of damping will lead respectively to unconservative or conservative results.

e) Nonlinear soil behaviour can not be explicity modelled in the frequency domain solutions used in elastic and visco-elastic half space representation of soil mass. But, the visco-elastic hysteretic model may be thought of as representing a limited degree of nonlinearity.

f) It is a essential to explore the application of a recent development by Bayo and Wilson (1984) which permits time domain solution of the interaction problem with much greater computational efficiency.

g) It is essential to make codal provisions for the interactive behaviour and develop some simplified procedures for interactive analysis useful in design offices.

h) As a performance study, there is a need to have better instrumentation which need be identified, installed and monitored on prototype structures.

i) There is a need to develop better experimental facilities like large size shake tables with simulated motion application and better monitoring facilities.

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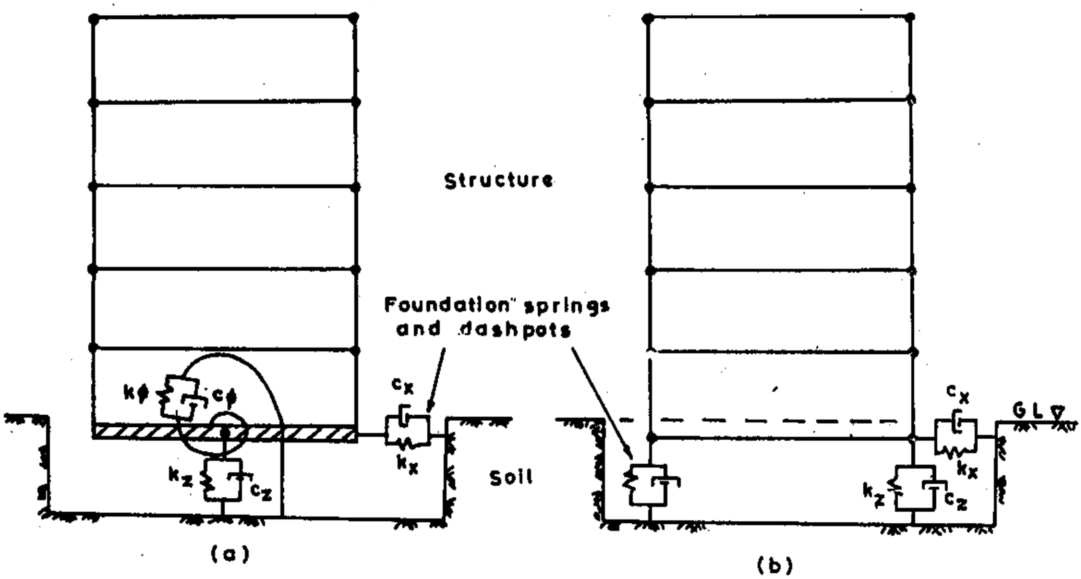


Fig.1 — Rudimentary soil-structure analytical models representing soil properties by spring and dashpot (Poulos and Devis-1974, Clough and Penzien-1975)

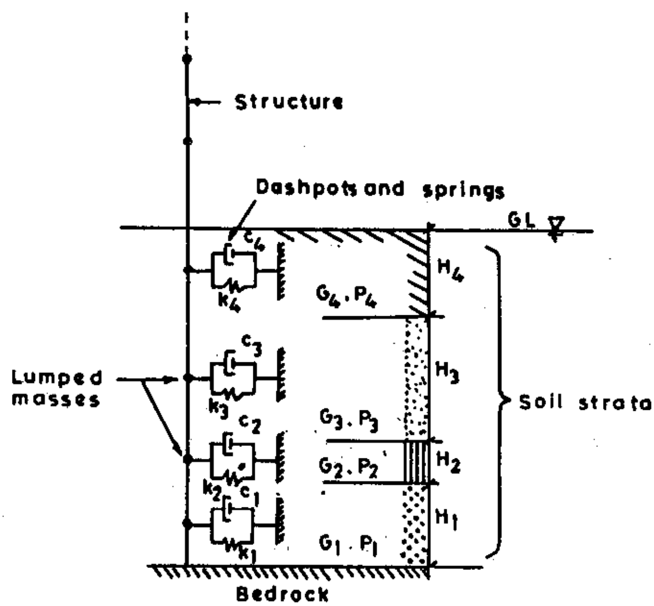


Fig.2 — Soil-structure analytical model representing the soil vertical profile by a lumped parameter system of masses, springs and dashpots (Penzien - 1970)

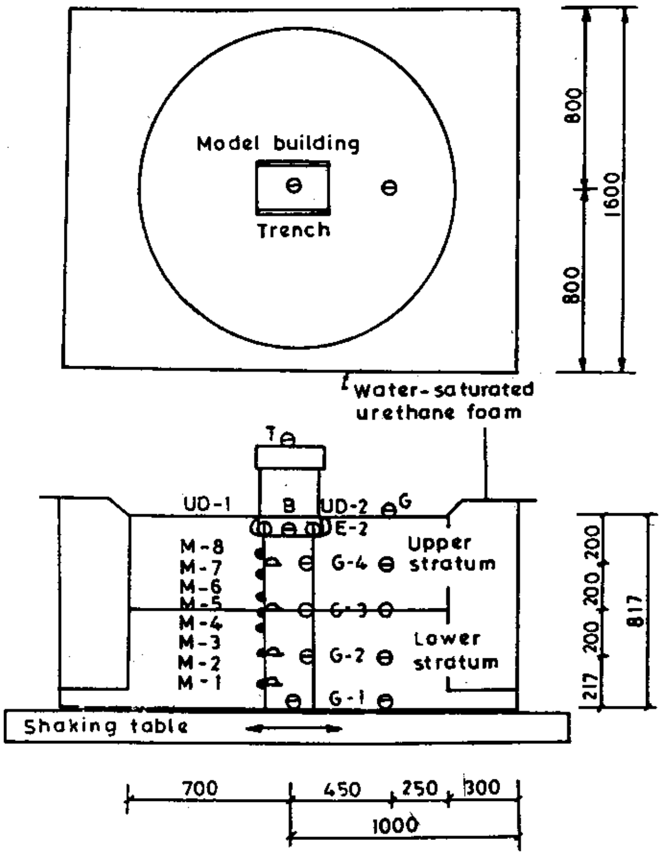


Fig.3– Experimental facilities and instrumentation (Mizuno et.al. ,1985)

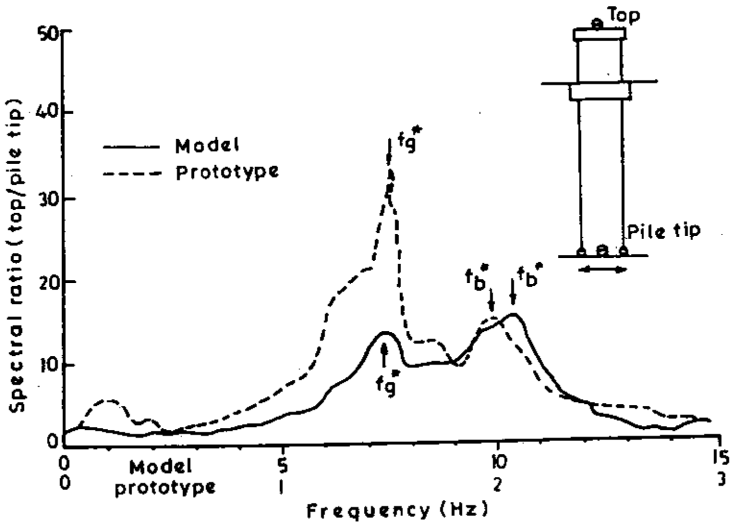


Fig. 4– Comparison of fourier spectral ratio (Building/pile tip)

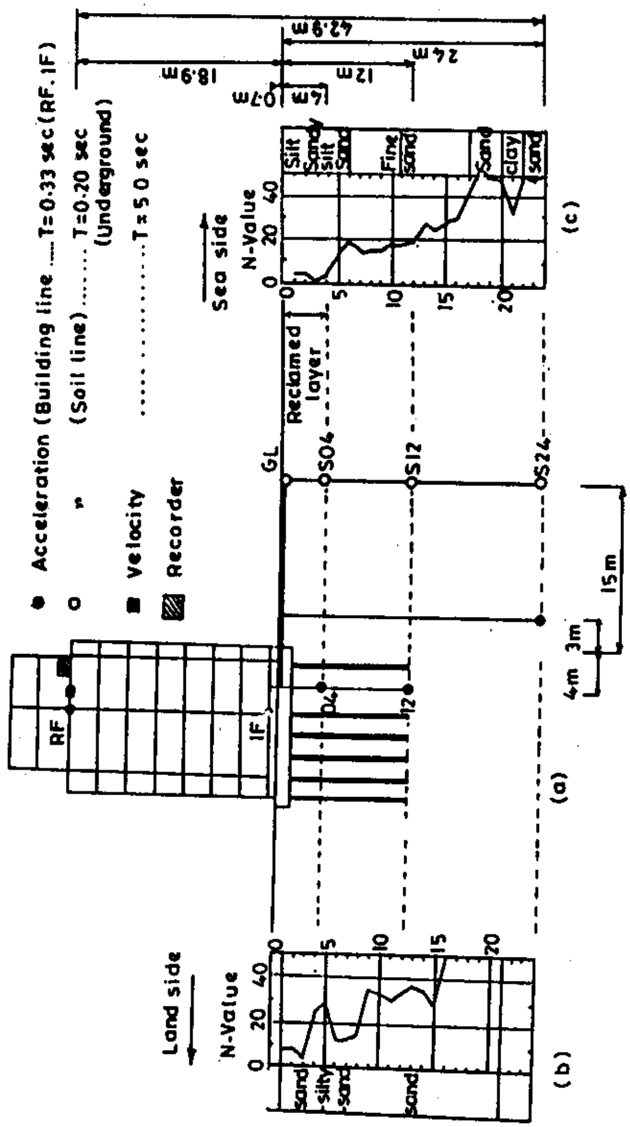


Fig. 5 — Pick-up location and soil profile