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WAVE IMPEDING EFFECT BY BURIED RIGID BLOCK AND RESPONSE REDUCTION OF DYNAMICALLY EXCITED PILE FOUNDATION

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Presented here is a verification of the effect of a buried block (termed as "wave impeding block"), when compared with a trench-cut measure, in reducing the response of a machine foundation in soft soils supported by piles and of the nearby soils. An annular type block whose depth is determined based on the wave cut-off phenomenon is devised to surround the piles. Steady state harmonic response is investigated for the horizontal, vertical and rocking exertion in the frequency range of interest. The numerical computation is conducted by the finite element method for an axisymmetric model under axisymmetric/anti-axisymmetric loading conditions.

Key Words : *wave impeding effect, FEM, transmitting boundary, ring pile analysis*

1. INTRODUCTION

Machine foundations transmit vibrations through ground to the nearby field, causing disturbance to it. Conventionally, the counter-measures have been devised to reduce the ground-transmitted vibration from the source through installation of barriers in the form of trenches. For the 2-dimensional problem, the wave screening effect by such measures are investigated theoretically by the finite element method by Lysmer and Waas (1972)[1] and by the boundary element method by Beskos, Dasgupta and Vardoulakis (1986)[2]. Installing a concrete wall was proposed by Haupt (1986)[3] and a wall of air cushions by Winkenholm and Agren (1986)[4]. Recently, an idea has been put forth by Chouw, Le and Schmid (1991)[5] to build a rigid block in the ground at a proper depth, depending on the frequency range below the cut-off frequency of wave propagation in the surface layer above it. Based on the investigation for a surface rigid footing, they reported its effectiveness and concluded it was a better engineering practice than cutting trenches. The cut-off frequency is determined from the shear wave velocity for horizontal motion and from the pressure wave velocity for vertical and rocking motions, from which the depth of the rigid block placement is properly determined. The transient response has recently been dealt with by Takemiya and Fujiwara (1993)[6], by performing extensive parameter studies in time domain to prove such impeding phenomenon.

Machine foundations in soft soils are usually supported by piles. In the case of end-bearing piles, the presence of a rigid base has opposite directional effects depending on the frequency range concerned ; in the frequency range below the cut-off frequency no wave transmission occurs, while in the frequency range above it the resonance possibility comes out due to the layer's eigenmodes as Chouw and Schmid (1991)[7] showed. The through understanding of wave transmission and impeding leads to an advanced rational design procedure for the foundation. Herein, attention is focused on such pile foundations for the development of a response reduction system with a modified design and construction of the wave impeding block. For this theoretical investigation, the finite element approach with use of transmitting boundary at the side of the model is used and the steady state response is presented herein. From the convenience of treating the 3-dimensional wave field, the axisymmetric 3-dimensional model is employed for axisymmetric and anti-axisymmetric harmonic loading on the foundation. The elaborate transient response analysis has been conducted in the authors' companion paper (1993)[8].

From the seismic design point of view, the effective foundation input motions and the nearby soil surface response are also important. These are evaluated herein for incident SV waves at the eigenfrequencies of the soil medium in order to measure the effect of the above wave impeding block since the rigidity of the block works for reducing the seismic input motion whereas the inertia may induce some resonance.

2. FORMULATION

Applying the substructure concept to the analysis of piles and nearby soils, one may superimpose the pile stiffness on to that of the soils for the

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interaction system. However, since the free field is used for the latter analysis, due care should be taken to evaluate the reduced piles stiffness by that of soils. The same holds true in assembling the mass matrix as well. The hysteric damping effect is taken into account for the internal energy dissipation in addition to the radiating wave horizontally towards infinity. The axisymmetric arrangement of piles is assumed for the model. Pile tips are built in to a presumed rigid bedrock and the pile heads are perfectly connected to a rigid body footing. The infinitely extending side region of the soil is replaced by the so-called transmitting boundary element based on the surface wave propagation (generalized Rayleigh and Love waves) in the concerned layer (Kausel and Roesset, 1975)[9]. Only some essence of the formulation is presented here from the work by Takemiya (1986)[10] to make the procedure clear.

In case of the concentric piles layout in plan view, the pile-soil-pile interaction may well be interpreted by the Fourier series expansion for the response description. Normally, a limited Fourier terms are used so as to meet the displacement compatibility for the footing motion. Specifically, for the vertical motion of a rigid footing, the $n=0$ axisymmetric term ; for the coupled sway along the x-axis and rocking motion about the y-axis of a rigid footing, the $n=1$ symmetric term about the x-axis.

The FEM modeling is performed by solid elements for the nearby soil and by the beam elements for the piles. The interaction between these elements are taken into account at the nodes only, demanding the nodal displacement compatibility and the corresponding forces equilibrium. The internal forces of piles is presumed to follow as mentioned above along the azimuth around the axisymmetric footing center.

The governing equation of motion of nearby soils is established in the cylindrical coordinates by expanding displacements and the forces into the Fourier series along circumferential direction.

$$U(r,\theta,z)=\sum_n H_n^s(\theta)U_n^s(r,z)+\sum_n H_n^a(\theta)U_n^a(r,z) \quad (1)$$

$$P(r,\theta,z)=\sum_n H_n^s(\theta)P_n^s(r,z)+\sum_n H_n^a(\theta)P_n^a(r,z) \quad (2)$$

in which the Fourier amplitude vectors $U_n(r,\theta,z)$, $P_n(r,\theta,z)$ with superscript "s" stand for symmetric Fourier components and "a" for anti-symmetric ones, the displacement and force vectors are

$$U(r,\theta,z)=(u_r \quad u_\theta \quad u_z)^T \quad (3)$$

$$P(r,\theta,z)=(p_r \quad p_\theta \quad p_z)^T \quad (4)$$

and the variation $H_n(\theta)$ along the circumferential direction is prescribed by the diagonal matrices of

$$H_n^s(\theta)=\mathrm{diag.}(\cos n\theta \quad -\sin n\theta \quad \cos n\theta) \quad (5.a)$$

$$H_n^a(\theta)=\mathrm{diag.}(\sin n\theta \quad \cos n\theta \quad \sin n\theta) \quad (5.b)$$

The finite element discretization is conducted by the conventional isoparametric assumption, leading the governing equation for the Fourier amplitudes as

$$\left[-\omega^2 M_{soil,n}^{s(a)}+i\omega C_{soil,n}^{s(a)}+K_{soil,n}^{s(a)}\right]U_{soil,n}^{s(a)}=P_{soil,n}^{s(a)} \quad (6.a)$$

or $\quad D_{soil,n}^{s(a)}U_{soil,n}^{s(a)}=P_{soil,n}^{s(a)} \quad (6.b)$

where $D_{soil,n}^{s(a)}$ defines the dynamic stiffness of $D_{soil,n}^{s(a)}=-\omega^2 M_{soil,n}^{s(a)}+i\omega C_{soil,n}^{s(a)}+K_{soil,n}^{s(a)}$ with $M_{soil,n}^{s(a)}$, $C_{soil,n}^{s(a)}$ and $K_{soil,n}^{s(a)}$ being the associated mass, damping and stiffness matrices, respectively ; ω is the frequency concerned.

The pile's equation, on the other hand, needs rotation angles in addition to displacements for describing the deformation and the associated forces including moments.

$$U(r,\theta,z)=(u_r \quad u_\theta \quad u_z \quad \Phi_r \quad \Phi_\theta \quad \Phi_z)^T \quad (7)$$

$$P(r,\theta,z)=(p_r \quad p_\theta \quad p_z \quad M_r \quad M_\theta \quad M_z)^T \quad (8)$$

The corresponding Fourier expansion in this case becomes as

$$\begin{Bmatrix} u(r,\theta,z) \\ \Phi(r,\theta,z) \end{Bmatrix}=\sum_n \begin{bmatrix} H_n^s(\theta) & \\ & H_n^a(\theta) \end{bmatrix}\begin{Bmatrix} u_n^s(r,z) \\ \Phi_n^s(r,z) \end{Bmatrix}+\sum_n \begin{bmatrix} H_n^a(\theta) & \\ & H_n^s(\theta) \end{bmatrix}\begin{Bmatrix} u_n^a(r,z) \\ \Phi_n^a(r,z) \end{Bmatrix} \quad (9)$$

$$\begin{Bmatrix} p(r,\theta,z) \\ M(r,\theta,z) \end{Bmatrix}=\sum_n \begin{bmatrix} H_n^s(\theta) & \\ & H_n^a(\theta) \end{bmatrix}\begin{Bmatrix} p_n^s(r,z) \\ M_n^s(r,z) \end{Bmatrix}+\sum_n \begin{bmatrix} H_n^a(\theta) & \\ & H_n^s(\theta) \end{bmatrix}\begin{Bmatrix} p_n^a(r,z) \\ M_n^a(r,z) \end{Bmatrix} \quad (10)$$

The governing equations of piles for the Fourier amplitudes can be obtained by the conventional finite element procedure based on the beam theory and are expressed in a similar form with Eq.(6). Therefore, for respective Fourier terms, the nodal displacement compatibility and the nodal force equilibrium constitute the pile and soil interaction equation.

$$(D_n^{s(a)}+R_n)U_n^{s(a)}=P_n^{s(a)} \quad (11)$$

in which $D_n^{s(a)}$ defines the total dynamic stiffness of pile-soil system within finite element domain, and the R_n impedance matrix of the lateral infinite soil domain on transmitting boundary that is estimated through three dimensional thin layer element method.

The foundation motion U_F at a reference node (master node) is expressed in the Cartesian coordinates. The transformation is therefore executed through $T_n^{s(a)}$, as a rigid body to the displacements at nodes of the soil-foundation interface (slave nodes). Thus, Eq.(11) results in

$$P_{s,n}^{s(a)} = (D^{s(a)} + R)_{c,n} U_{s,n}^{s(a)} = (D^{s(a)} + R)_{c,n} T_n^{s(a)} U_F \quad \cdots\cdots (12)$$

in which $(D^{s(a)}+R)_{c,n}$ defines the dynamic stiffness after condensing out the degrees of freedom in $(D^{s(a)}+R)_n$ except those at slave nodes ; $P_{s,n}^{s(a)}$ is the corresponding condensed driving force.

Furthermore, the soil-foundation interface nodal forces including the moment in the cylindrical coordinates are also transformed into those for the master node of the foundation in the Cartesian coordinates

$$P_F = \sum_n \alpha_n (T_n^{sT} P_{s,n}^s + T_n^{aT} P_{s,n}^a) \quad \cdots\cdots (13)$$

with $\alpha_n = 2\pi$ for $n=0$ and oterwise. It follows from Eqs.(12), (13) that we get the equilibrium equation of the footing as

$$[\sum_n \alpha_n \{T_n^{sT}(D^s+R)_{c,n} T_n^s + T_n^{aT}(D^a+R)_{c,n} T_n^a\} - \omega^2 M_F] U_F = P_{sup} + P_g \quad \cdots\cdots (14)$$

in which P_g is the effective input force due to the kinematic interaction with soils through seismic wave incidence and P_{sup} is the force to exert directly from the superstructure.

For incident SV plane wave, the displacement at base nodes is defined as

$$\left. \begin{aligned} u_{b,n}^{s(a)} &= A T_{b,n}^{s(a)} u_0 \\ u_0 &= (\cos\gamma \quad 0 \quad \sin\gamma)^T \\ A &= e^{ik_s \cos\gamma z_0} \end{aligned} \right\} \quad \cdots\cdots (15)$$

where u_0 is unit displacement vector, γ is incident angle, k_s is wave number and z_0 is the vertical coordinate of the base nodes. The transform matrix $T_{b,n}^{s(a)}$ can be determined by introducing Bessel functions to represent the plane wave propagation. Thus the free-field displacements and forces on lateral transmitting boundary can be calculated by one dimensional model appropriately, and then the effective input motion and/or force to foundation are determined.

3. NUMERICAL STUDIES

The wave transmission in a layer underlain by a rigid bedrock is mostly executed by the eigenmodes

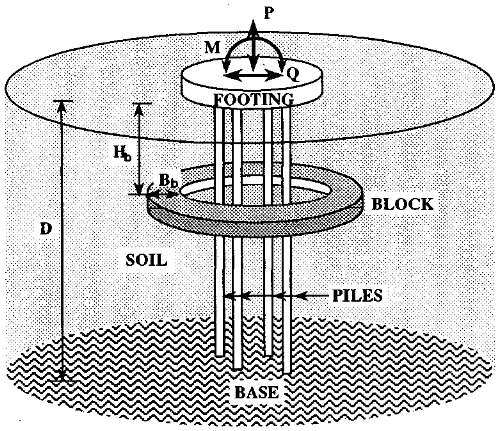

Fig.1 Installation of a wave impeding ring block

for body waves and surface waves. In the vicinity of the source the body waves dominates the soil response and only the surface waves remain as the observation distance is increased from the source because the body waves die out. The eigenvalues for the former wave propagation are calculated for the P and SV wave respectively, by

$$\left. \begin{aligned} f_{sc,s}^m &= \frac{V_s}{4D}(2m-1) \\ f_{sc,p}^m &= \frac{V_p}{4D}(2m-1) \\ m &= 1, 2, \cdots \end{aligned} \right\} \quad \cdots\cdots (16)$$

Since our primary focus is placed on the response of the near field as well as the footing, the wave cut-off frequency is specified approximately by the first eigenvalues. The depth H_b of a rigid block installation is therefore determined by

$$f_{bc,s}^1 = \frac{V_s}{4H_b}, \quad f_{bc,p}^1 = \frac{V_p}{4H_b} \quad \cdots\cdots (17)$$

In view of the axisymmetric nature of the model for analysis, a block to be installed bellow the foundation is formed annular to encompass the piles, as illustrated in **Fig.1**. The pile deformation is significant at the portion above the so-called $1/\beta$ from the Chang's formula, causing wave generation and propagation outwards. From this reason, the wave impeding block is devised as an annular to surround piles. Besides, filling in this inside space with the block material, which bonds the piles together in effect, is intentionally avoided in view of the high possibility of concentrated forces at the pile sections. **Fig.2** describes the pile foundation for analysis. The depth of an annular block is determined aiming at the wave impeding in the frequency range below 6 [Hz] for the horizontal loading and for the vertical loading in the frequency range between 4 [Hz] and 10 [Hz]. This requirement comes from the machine foundation

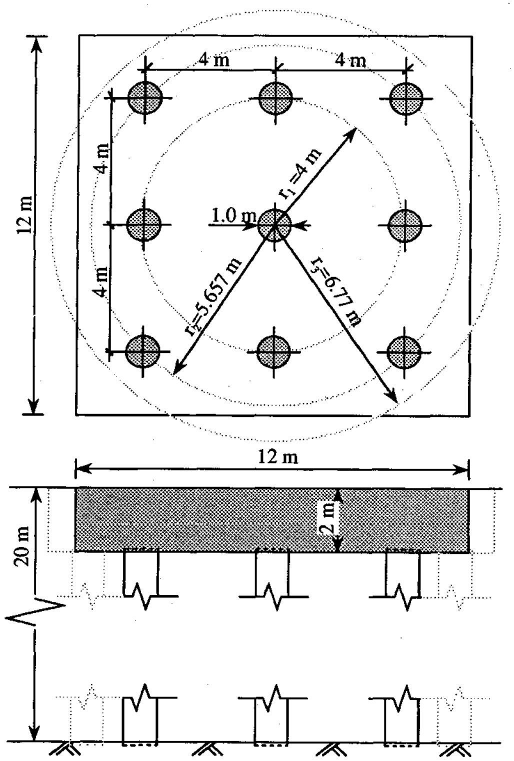

Fig.2 Pile foundation for analysis

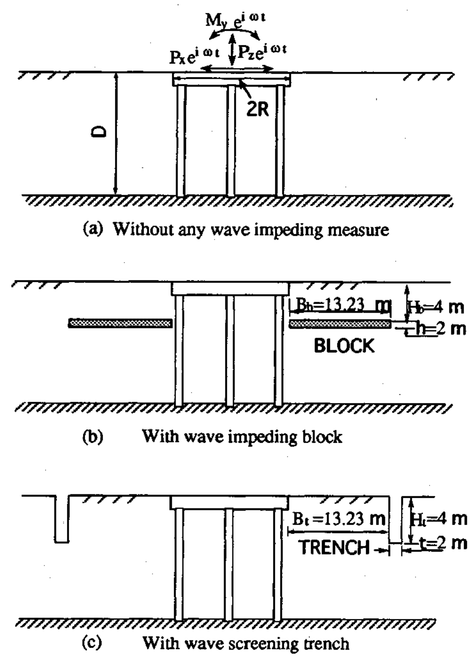

Fig.3 Model for numerical computation

Table 1 Material properties of soil, pile and block

property \ material	soil	pile	block
density ρ (t/m³)	1.8	2.4	2.4
Poisson ratio ν	0.4	0.167	0.167
damping ratio β (%)	3	0	0
shear wave velocity V_S (m/s)	100	-	2000
Young's modulus E (kn/m²)	-	2.1×10^7	-
section area A (m²)	-	0.785	-
section moment $I_x(=I_y)$ ($10^{-2}m^4$)	-	4.91	-
section moment I_z ($10^{-2}m^4$)	-	9.82	-

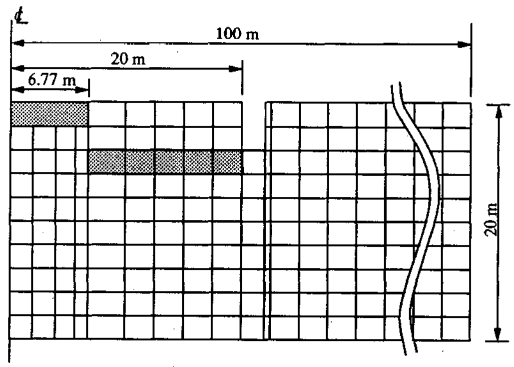

Fig.4 Finite element discretization

of interest. Simple calculation indicates that for the S wave the layer resonance frequencies are $f_n=$ 1.25, 3.75, 6.25 and 8.75 [Hz] in the ascending order, then the corresponding wave lengths are λ_n $=80$, 26.7, 16, 11.4 [m] ; for the P wave $f_n=3.06$, 9.19, 15.31 and 21.44 [Hz] which yield the same λ_n's. Hence, the location and the dimensions of the wave impeding block in **Fig.3**(b) result. Trench cut is also investigated for the wave screening in order to compare it with that of the present wave impeding block. Since there remain some uncertainties for deciding the proper size and distance of the trench-cut measure, here the depth is chosen the same as the above block location and the distance at the far end of it (**Fig.3**(c)). The material properties of the soils, piles and buried block are indicated in **Table 1**. **Fig.4** illustrates the models for FEM computation without any impeding measure, with a wave impeding block, and with a trench.

Fig.5. shows the radial and circumferential responses at the nearby soil surface due to a horizontal (x-direction) loading on the rigid massless footing. Normalized presentation is made by dividing the soil surface responses by the horizontal footing displacement. Analysis is made at the frequencies below and above the cut-off frequency $f^1_{bc,s}=6.25$ [Hz] for the S-wave propagation in the soil layer above the block, and the results are compared among models with the annular block installation, with a ring trench cut and without any wave impeding measures. One can

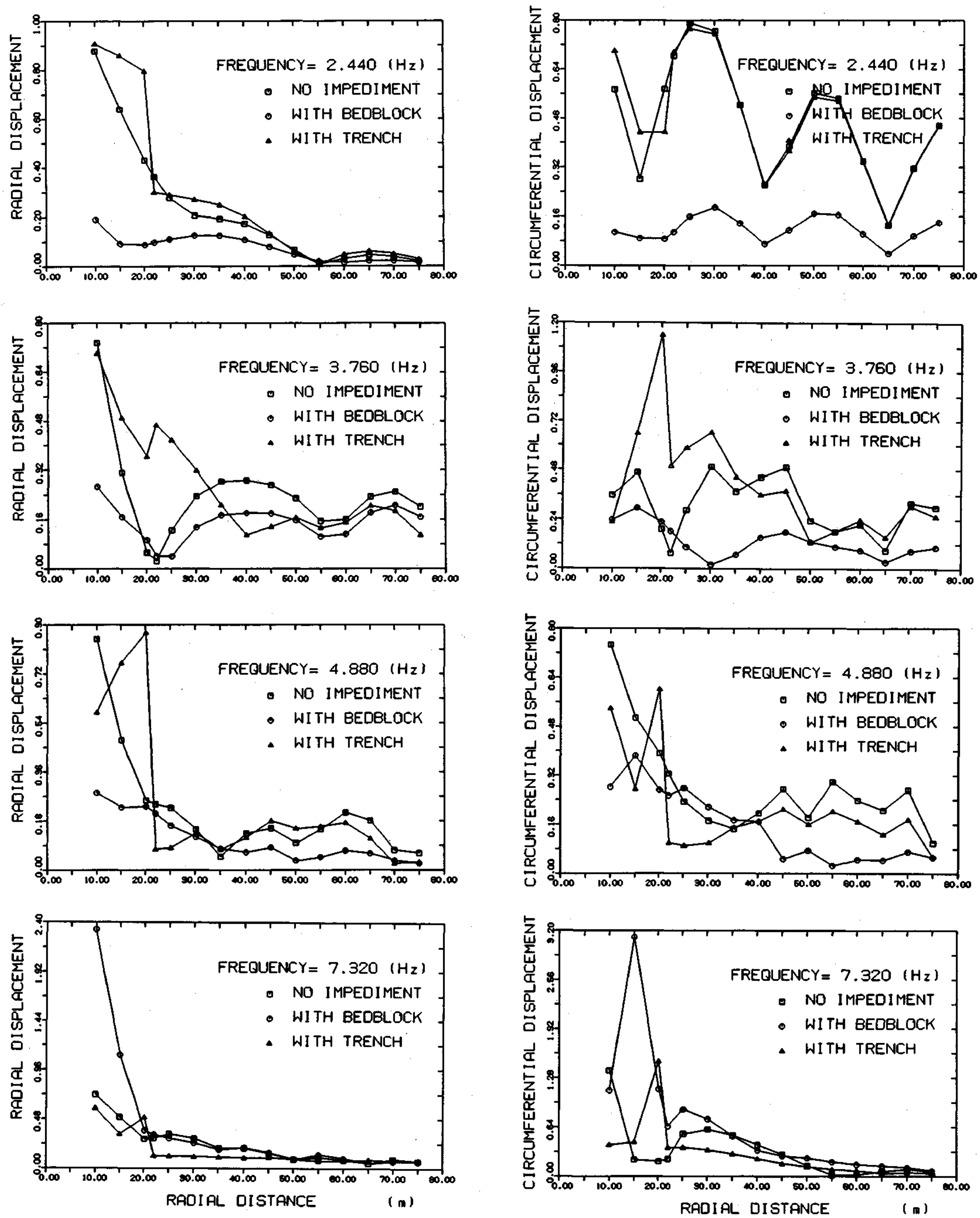

Fig.5 Normalized soil surface responses due to horizontal foundation excitation

note that for a frequency range below $f_{bc,s}^{1}$, a significant response reduction is attained by the annular block at the surface above it and the farther horizontal distance. However, for the higher frequency range beyond $f_{bc,s}^{1}$, the response amplification occurs as the layer's first eigenmode comes in this frequency range. The trench cut, on the other hand, leads to a substantial wave screening behind it but a significant unfavorable increase of response at the soil surface in front of it. The response feature at the far distance reflects the surface wave propagation in a horizontal direction : The radial response yields the generalized Rayleigh wave propagation while the circumferential response gives the generalized Love wave propagation.

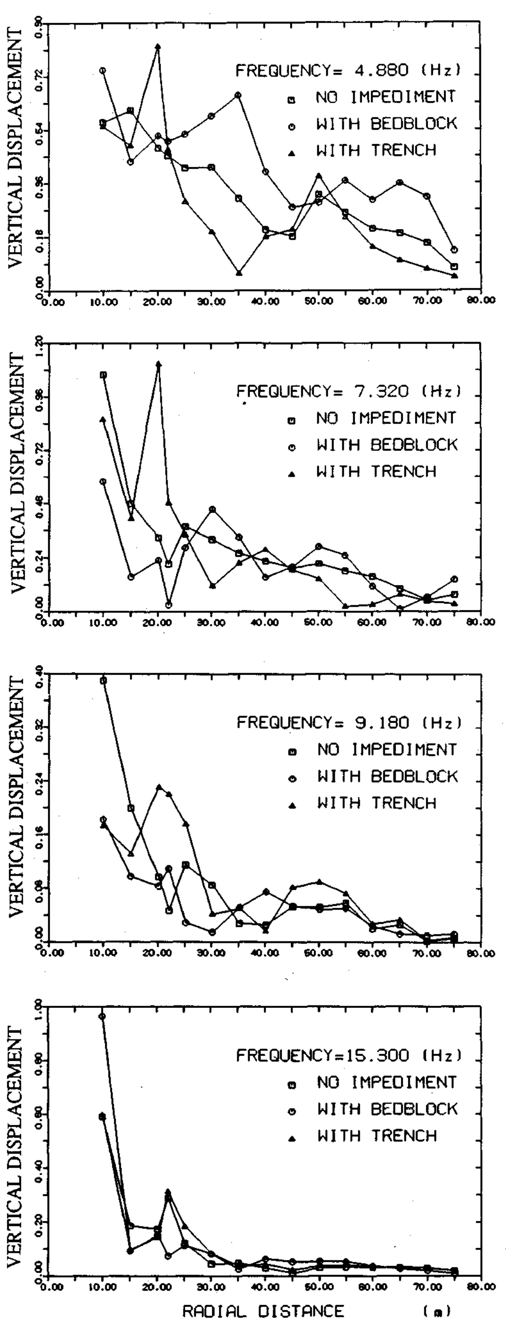

Fig.6 Normalized soil surface responses due to vertical foundation excitation

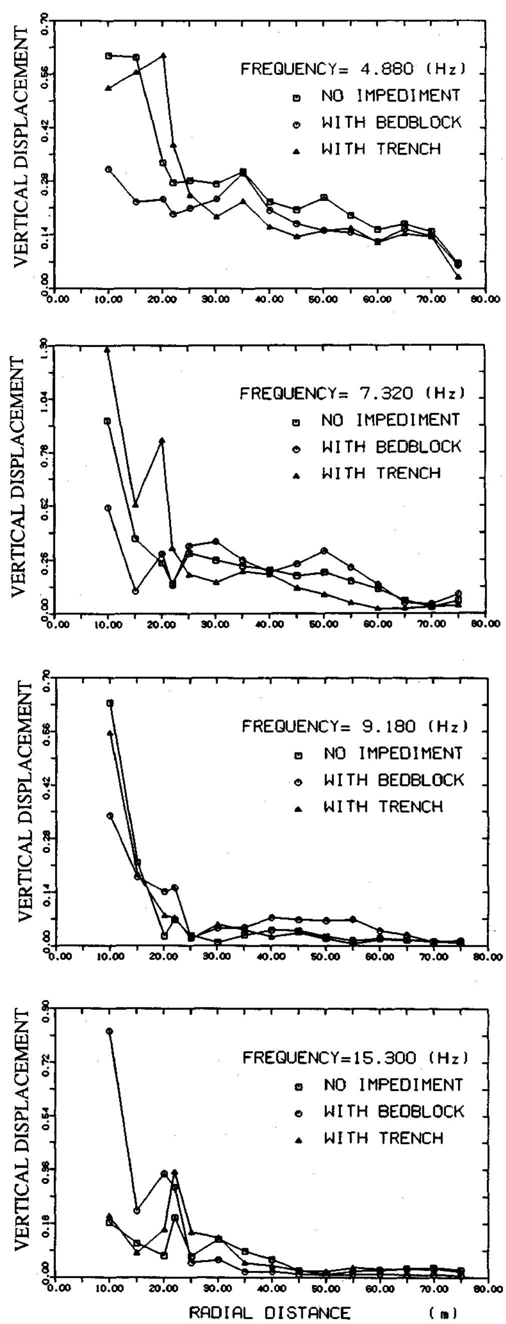

Fig.7 Normalized soil surface responses due to rocking foundation excitation

Fig.6 indicates the vertical response at soil surface due to a vertical (z-direction) loading on the massless footing. Normalization is executed by dividing it by the vertical footing displacement. The annular block fails in reducing the response of nearby soil in the low frequency range. The first eigenfrequency for the natural soil layer is given by $f^1_{sc,p}=3.06$ [Hz], which results in amplifying the response instead. However, as the frequency is increased beyond this value, the annular block works for reducing the nearby soil response in the frequency range below the cut-off frequency $f^1_{bc,p}=15.31$ [Hz] which is specified by the P-wave propagation in the soil layer above the block.

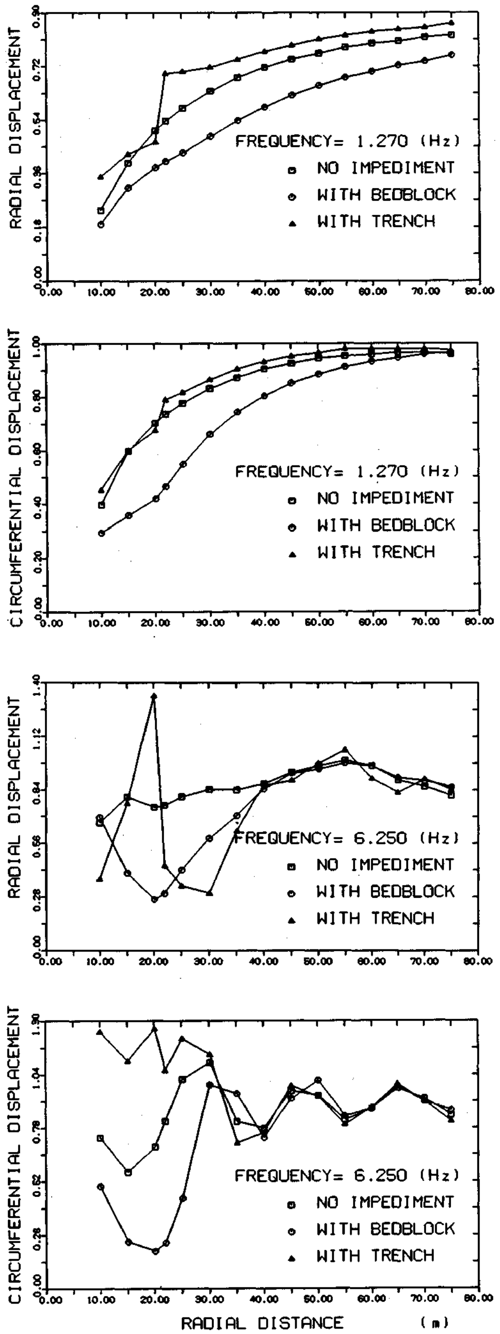

Fig.8 Normalized soil surface responses due to vertical incident SV wave

Fig.7 gives the vertical response at soil surface due to a moment loading on the rigid massless footing. Normalization is taken by dividing the soil response by the footing rotation multiplied by the radius. The rocking motion of the foundation is more concerned with the P-wave than the S-wave propagation of the soil layer so that the wave impeding effect of the annular block has the same tendency with the vertical response due to vertical loading.

Another important aspect of installing a rigid block in the soil medium is to what extent the ground motion of foundation and nearby soil surface is affected by it in the case of base excitation when the seismic motion impinges the bedrock. The effect of the block rigidity works for reducing the seismic input motion to the foundation, whereas the effect of the mass may induce some resonance. In this paper, the effective input motion through the kinematic interaction between soil and footing is considered for the plane SV motion. The computation results as normalized by the free field response are depicted in **Fig.8** for the radial, circumferential components at the eigenfrequencies of interest of the soils. It is worth to note that the installing the block reduces the ground motion significantly in the radial direction and also that in the circumferential direction.

4. CONCLUSION

Installation of a rigid block bellow a machine foundation, whose depth is to be determined in view of the cut-off frequency of a layer underlain by a rigid base (in fact the soil thickness above the block), has been proposed as a promising alternative measure for a wave impeding system in order to reduce the nearby soil response. Since the present study is oriented to a pile foundation at soft soils, an annular stiff block surrounding piles is devised. The investigation is conducted on the axisymmetric 3-dimensional model under the axisymmetric and anti-axisymmetric loading on the rigid massless footing. In comparison with cutting trenches, the numerical computation demonstrates that the annular block is more effective for reducing the soil response at nearby soils in the frequency range below the cut-off frequency. Installing the above block also reduces the foundation input motion at the soil resonance frequencies for incident seismic waves.

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動的載荷を受ける杭基礎の埋設剛ブロックによる波動遮断と応答制振性

竹宮宏和・蔣　建群

本論文は，杭支持された機械基礎の振動およびその周辺地盤動を低減させるための対策として円環状の埋設ブロック（ここでは波動遮断ブロックと呼ぶ）を層状地盤の波動遮断理論から設定した際の効果を検討したものである．解析は有限要素法で，特定の振動数帯域における定常調和振動状態において，軸対称モデルの軸対称／非軸対称加振の下で行った．