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DYNAMIC BEHAVIORS OF STRUCTURES DUE TO GROUND MOTIONS CONSIDERING PHASE DIFFERENCES

Kazushige YAMAMURA[1], Takao NISHIKAWA[2] and Ryuji NAKANISHI[3]

SUMMARY

To examine dynamic behaviors of structures due to ground motions considering phase differences, the elastic response analyses were conducted, in which rigidity of foundation slabs, shear wave velocity (rigidity of the ground), structural overall length were assumed to be parameters. SH waves inputted obliquely from perpendicular downward were considered as input ground motions.

As a result of the analyses, the dynamic responses of the structures due to ground motions considering phase differences were considered in terms of the relationship between rigidity of each spring of the structures, that of the ground, the incident angle, etc., and these relationships were examined quantitatively.

INTRODUCTION

In the structural design practice, input ground motions are usually treated as a body wave inputted vertically from perpendicular downward. But it is thought that actual ground motions are not always inputted vertically and sometimes inputted obliquely. When a ground motion is inputted obliquely, each part of a foundation of a structure is exerted by different forces.

In recent years, many structures with a large floor area are constructed. And it is thought that such structures are affected by oblique input ground motions. So the elastic response analyses are conducted in order to examine dynamic behaviors of structures due to ground motions considering phase differences.

ANALYTICAL MODEL

The unit model is the single-degree-of-freedom system which is supported by the foundation slab with the horizontal soil spring k_H. And the analytical model is the multi-degree-of-freedom system which consist

1 Research Associate, Tokyo Metropolitan University, Tokyo, Japan. Email: kyamamur@arch.metro-u.ac.jp
2 Professor, Tokyo Metropolitan University, Tokyo, Japan. Email: tanishi@arch.metro-u.ac.jp
3 Japan Nuclear Cycle Development Institute, Ibaraki, Japan. Email: nakanishi.ryuuji@jnc.go.jp

of the unit models serially connected by the floor slab spring k_s and the foundation slab spring k_f. Fig.1 shows the analytical model. In order to define parameters of the model, the followings are assumed.

1. The size of the unit structure is 12m for span direction and 8m for longitudinal direction.
2. A natural period of the unit structure is about 0.3 seconds.
3. Mass of the unit structure is 1ton/m^2 and mass of the foundation is 3 times as heavy as the unit structure.
4. The floor slab is a RC slab with 12cm thick and the stiffness of the slab spring is estimated based on in-plane deformation.
5. The horizontal soil spring is estimated by the Parmelee's equation. (where poisson ratio=0.3, density=1.5t/m^3)
6. Damping of the system is local viscous damping. The damping factor of the upper structure and the foundation is 0.02 and 0.1 respectively.

The other parameters have several values as shown in Tab.1.

SH waves inputted obliquely from perpendicular downward were considered as input ground motions. Because of the oblique angle of incidence, the input ground motions to each supporting point have phase differences (time differences). Fig.2 shows the direction of input ground motions and an angle of incidence θ.

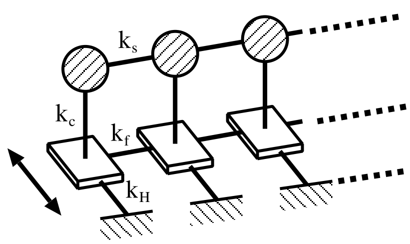


Fig.1 Analytical model

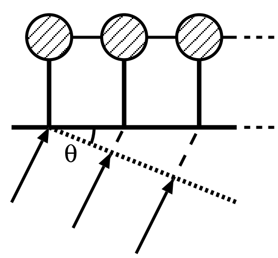


Fig.2 Input ground motion

Tab.1 Parameters

ℓ : overall length of structure	24m(3spans), 48m(6spans), 96m(12spans), 192m(24spans)
k_f / k_s : ratio of stiffness of slabs	0, 1, 2, 4, 8, 16, 32
V_S : velocity of shear wave	100m/s, 400m/s
θ : angle of incidence	0°, 6°, 12°,18°,24°,30°

EQUATION OF VIBRATION

Because of the effect of the floor and foundation slab springs, vibration of the model is express by the next equation.

$$[M]\{\ddot{y}\}+[C]\{\dot{y}\}+[K]\{y\}=-[M]\{\ddot{y}_0\}+[K_G]\{y_0\} \tag{1}$$

where $[M]$ is mass matrix, $[C]$ is damping matrix, $[K]$ is stiffness matrix, $\{y\}$ is vector of displacement of the structure, $\{y_0\}$ is vector of displacement of the ground and $[K_G]$ is matrix related to forces caused by the ground displacement.

When the model consists of three unit structures, $[K]$ and $[K_G]$ are expressed as follows.

$$[K]=\begin{bmatrix} k_c+k_s & -k_s & 0 & -k_c & 0 & 0 \\ -k_s & k_c+2k_s & -k_s & 0 & -k_c & 0 \\ 0 & -k_s & k_c+k_s & 0 & 0 & -k_c \\ -k_c & 0 & 0 & k_H+k_c+k_f & -k_f & 0 \\ 0 & -k_c & 0 & -k_f & k_H+k_c+2k_f & -k_f \\ 0 & 0 & -k_c & 0 & -k_f & k_H+k_c+k_f \end{bmatrix} \quad (2)$$

$$[K_G]=\begin{bmatrix} -k_s & k_s & 0 & 0 & 0 & 0 \\ k_s & -2k_s & k_s & 0 & 0 & 0 \\ 0 & k_s & -k_s & 0 & 0 & 0 \\ 0 & 0 & 0 & -k_f & k_f & 0 \\ 0 & 0 & 0 & k_f & -2k_f & k_f \\ 0 & 0 & 0 & 0 & k_f & -k_f \end{bmatrix} \quad (3)$$

Vector of displacement of ground motions with phase difference θ can be express by the next equation.

$$\{y_0\}^T = a_0 e^{ipt}\{e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots \quad e^{-(n/2-1)i\theta} \quad e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots\} \quad (4)$$

Then acceleration is,

$$\{\ddot{y}_0\}^T = -p^2 a_0 e^{ipt}\{e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots \quad e^{-(n/2-1)i\theta} \quad e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots\} \quad (5)$$

If the right side of eq(1) is expressed as $\{f\}$,

$$\{f\} = -[M]\{\ddot{y}_0\} + [K_G]\{y_0\} \quad (6)$$

Substituting eq(4) and (5) into (6),

$$\{f\} = p^2 a_0\left[[M] + \frac{1}{p^2}[K_G]\right]\{e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots \quad e^{-(n/2-1)i\theta} \quad e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots\}^T e^{ipt} \quad (7)$$

When $\{y\}$ can be expressed by the sum of product of s-th mode vector $\{_s u\}$ and coefficient $_s q$ as follows,

$$\{y\} = \sum_{s=1}^{n} \{{}_s u\}_s q \qquad (8)$$

And variable ${}_s G$ is defined by the next equation,

$${}_s G = \{{}_s u\}\left[[M] + \frac{1}{p^2}[K_G]\right]\{e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots \quad e^{-(n/2-1)i\theta} \quad e^{-0i\theta} \quad e^{-i\theta} \quad e^{-2i\theta} \quad \cdots\}^T \qquad (9)$$

$\{y\}$ is,

$$\{y\} = \sum_{s=1}^{n} \{{}_s u\} \frac{1}{{}_s\omega^2 - p^2 + 2{}_s h_s \omega p i} \frac{{}_s G}{{}_s M} p^2 a_0 e^{ipt} \qquad (10)$$

where ${}_s\omega$ is s-th natural circular frequency, ${}_s h$ is s-th mode damping factor and ${}_s M$ is s-th generalized mass.

Considering the real part of eq(10), displacement is expressed the next equation.

$$\{y\} = \sum_{s=1}^{n} \left(\left({}_s A \operatorname{Re}_s G - {}_s B \operatorname{Im}_s G \right) \cos pt - \left({}_s B \operatorname{Re}_s G + {}_s A \operatorname{Im}_s G \right) \sin pt \right) \{{}_s u\} \frac{p^2 a_0}{{}_s K} \qquad (11)$$

where ${}_s A = \dfrac{1 - \left(\dfrac{p}{{}_s\omega}\right)^2}{\left\{1 - \left(\dfrac{p}{{}_s\omega}\right)^2\right\}^2 + 4{}_s h^2 \left(\dfrac{p}{{}_s\omega}\right)^2}$, ${}_s B = \dfrac{-2{}_s h \left(\dfrac{p}{{}_s\omega}\right)}{\left\{1 - \left(\dfrac{p}{{}_s\omega}\right)^2\right\}^2 + 4{}_s h^2 \left(\dfrac{p}{{}_s\omega}\right)^2}$

EIGENVALUE ANALYSIS

Based on eq(1), (2) and (3), eigenvalue analyses are conducted. In order to estimate effects of soil-structure interaction, eigenvalues of another model without soil spring are also calculated. Tab.2 shows the results. Regardless of the overall length of structure ℓ, natural periods of 1st mode is almost same for every unit model. But natural periods of 2nd and 3rd mode become longer as the length is larger. The natural periods of SDOF mode are same as those of V_s =400m/s model and shorter than those of V_s=100m/s model.

Characteristics of modal damping factor h are different for every model. Some of participation factors β are 0 which means that no vibration occur in such mode by uniform input motions.

A mode shape of 1st mode is a translational motion of upper structures, that of 2nd mode is its motion with one node and that of 3rd mode is with two nodes. In few of the case mode shapes is different as mentioned before.

Tab.2 Natural period, damping factor and participation factor

ℓ	mode	SDOF			V_S = 400m/s			V_S = 100m/s		
		T[s]	h	β	T[s]	h	β	T[s]	h	β
24m	1st	0.315	0.020	2.0	0.316	0.021	2.1	0.334	0.031	2.5
	2nd	0.068	0.093	0.0	0.068	0.093	0.0	0.142	0.209	1.8
	3rd	0.038	0.166	0.0	0.038	0.806	1.9	0.068	0.098	0.0
48m	1st	0.315	0.020	2.6	0.316	0.021	2.7	0.335	0.032	3.3
	2nd	0.112	0.056	0.0	0.112	0.056	0.0	0.151	0.195	2.4
	3rd	0.061	0.103	0.0	0.061	0.104	0.0	0.113	0.060	0.0
96m	1st	0.315	0.020	3.6	0.316	0.021	3.7	0.336	0.032	4.6
	2nd	0.182	0.035	0.0	0.182	0.035	0.0	0.183	0.038	0.0
	3rd	0.105	0.060	0.0	0.105	0.060	0.0	0.157	0.187	3.2
192m	1st	0.315	0.020	5.0	0.316	0.021	5.1	0.336	0.032	6.5
	2nd	0.253	0.025	0.0	0.254	0.025	0.0	0.261	0.031	0.0
	3rd	0.177	0.036	0.0	0.177	0.036	0.0	0.178	0.039	0.2

STATIONARY RESPONSE TO HARMONIC INPUTS

Based on eq(11), stationary responses to harmonic inputs are calculated. To compare with the following results, amplification factor of SDOF and sway system are shown in Fig.3. Characteristics of SDOF model and V_s=400m/s model is almost same and the maximum value of amplification factor is about 25. V_s=100m/s model has longer peak period and the maximum value is smaller than others.

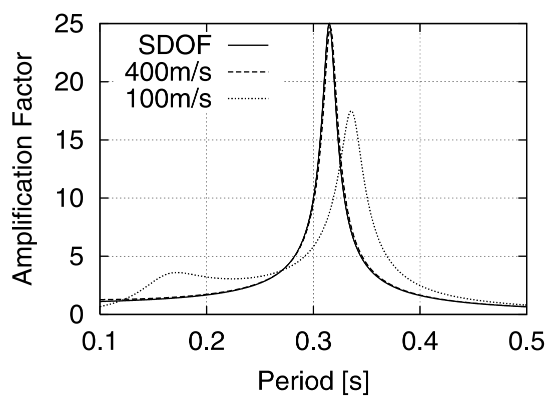
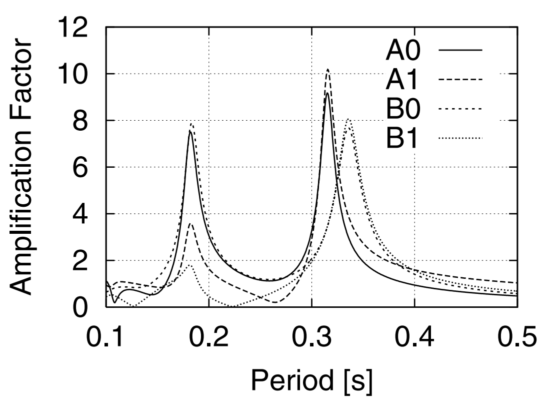


Fig.3 Amplification factor of SDOF and sway system

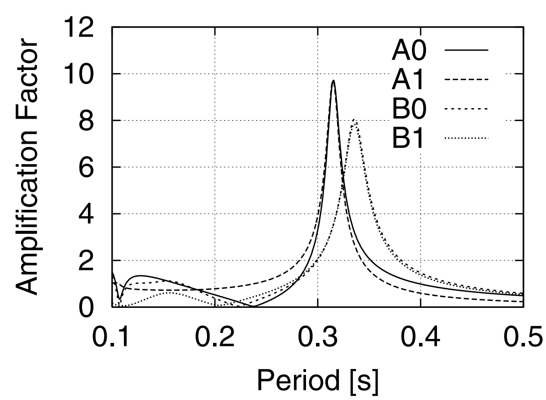
Fig.4 shows the effects of ground displacement. V_s=100m/s, θ=12degree, k_f / k_s=8 and ℓ=96m are assumed. Model A0 means the connected SDOF model not considering ground displacement (in eq(1) $\{y_0\}$ is assumed $\{0\}$), A1 means the connected SDOF model considering ground displacement, B0 means the connected sway model not considering ground displacement and B1 means the connected sway model considering ground displacement. When ground displacement is not considered, amplification factor of 2nd mode tends to be estimated too large. So it is important to consider ground displacement in the phase difference analyses. But the central lumped mass seems not to be affected by phase difference.

Fig.5 and 6 show the effects of phase difference. ℓ=192m and k_f / k_s=8 are assumed. When the angle of incidence is large, amplification factor of 1st mode is small and that of 2nd mode is large. And this tendency is obvious at the left side lumped mass and on the assumption of V_s=400m/s. Same as Fig.4, the central lumped mass is not affected by phase difference.

Fig.7 and 8 show the effects of overall length of a structure. k_f / k_s=8 and θ=24° are assumed. Only in the 192m model, the effects of 2nd and above mode are obvious and the amplification factor of 1st mode is very small.

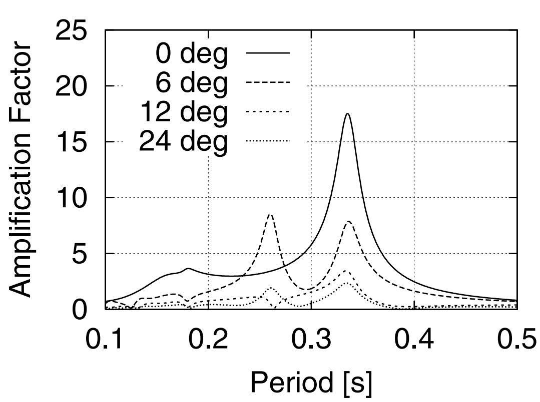


left side lumped mass

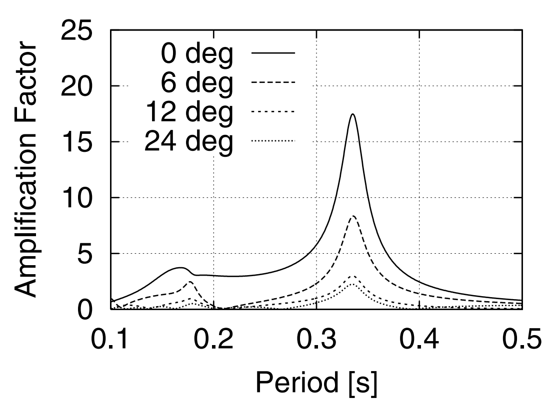


central lumped mass

Fig.4 Effects of ground displacement

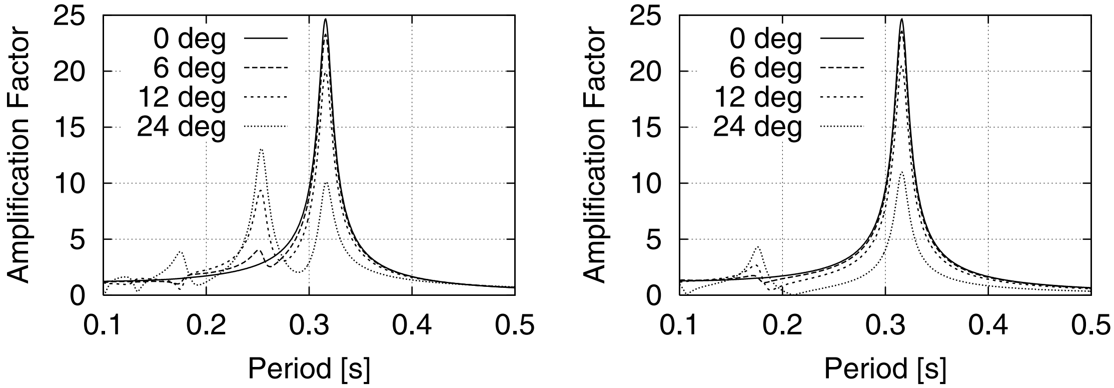


left side lumped mass



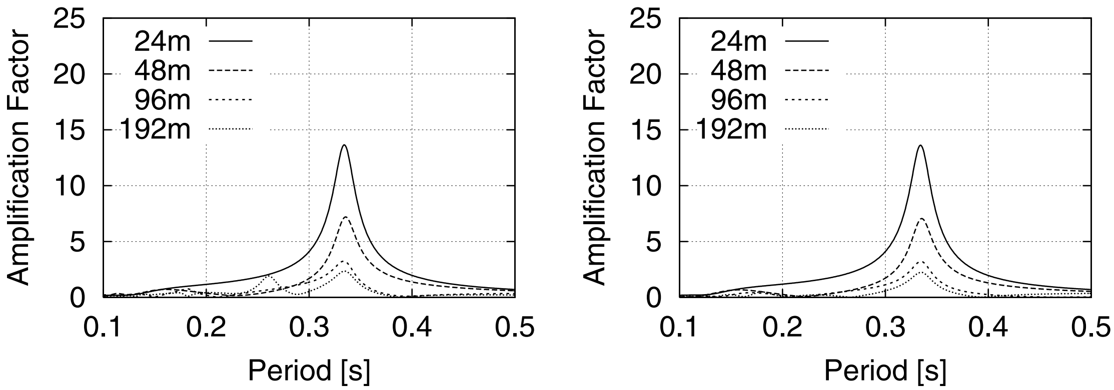
central lumped mass

Fig.5 Effects of phase difference (V_s=100m/s)



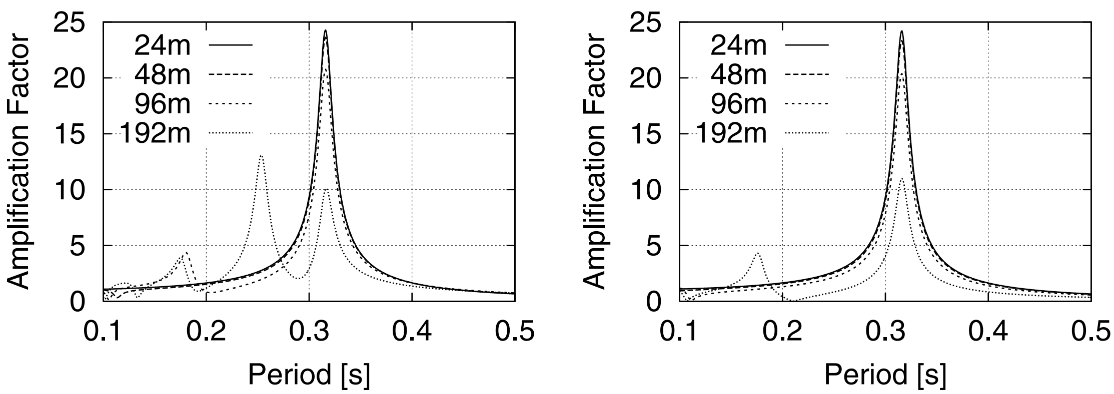
left side lumped mass   central lumped mass

Fig.6 Effects of phase difference (V_s =400m/s)



left side lumped mass   central lumped mass

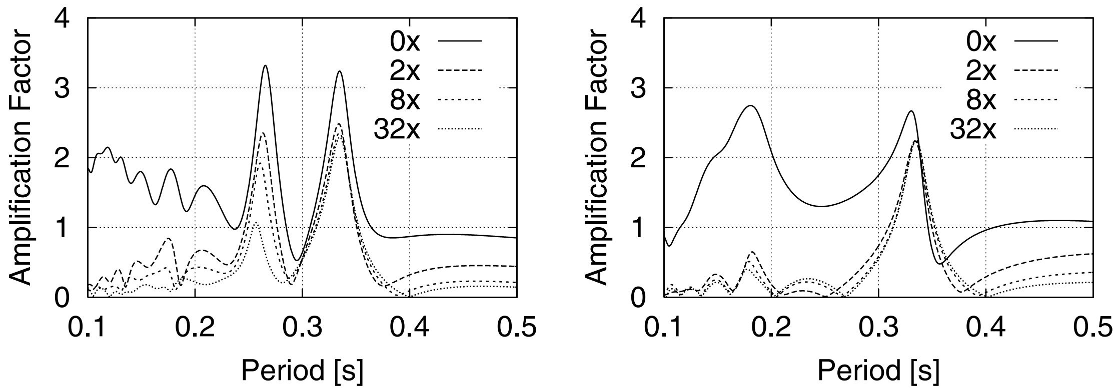
Fig.7 Effects of overall length of a structure (V_s =100m/s)



left side lumped mass   central lumped mass

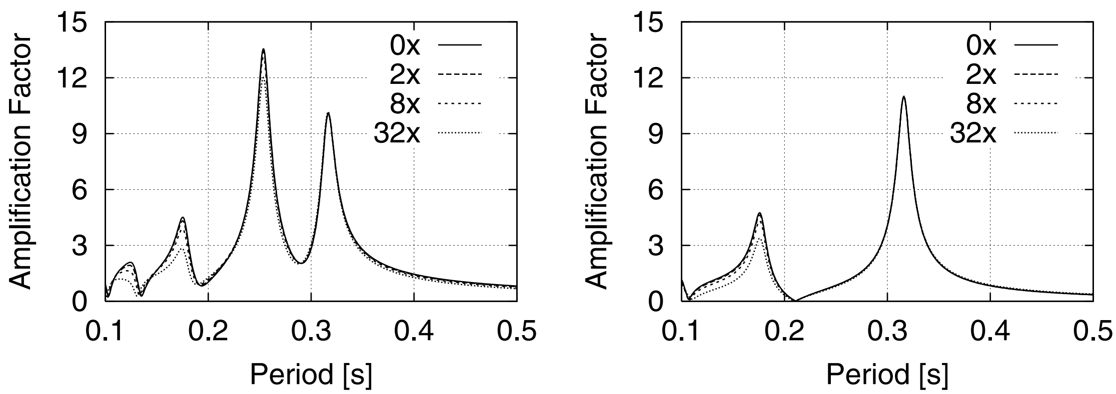
Fig.8 Effects of overall length of a structure (V_s =400m/s)

Fig.9 and 10 show the effects of stiffness of foundation slabs. ℓ=192m and θ=24° are assumed. In the case of V_s=400m/s, values of k_f / k_s do not affect the amplification factor. But in the case of V_s=100m/s, the amplification factor show the complex tendency in the range of shorter periods. When the shear wave velocity is slow, phase difference of harmonic waves with shorter period becomes very huge. It is the reason of such complex tendency. It is thought that this tendency affects the earthquake response of structures very much.



left side lumped mass central lumped mass

Fig.9 Effects of stiffness of foundation slabs (V_s=100m/s)



left side lumped mass central lumped mass

Fig.10 Effects of stiffness of foundation slabs (V_s=400m/s)

EATHQUAKE RESPONSE ANALYSIS

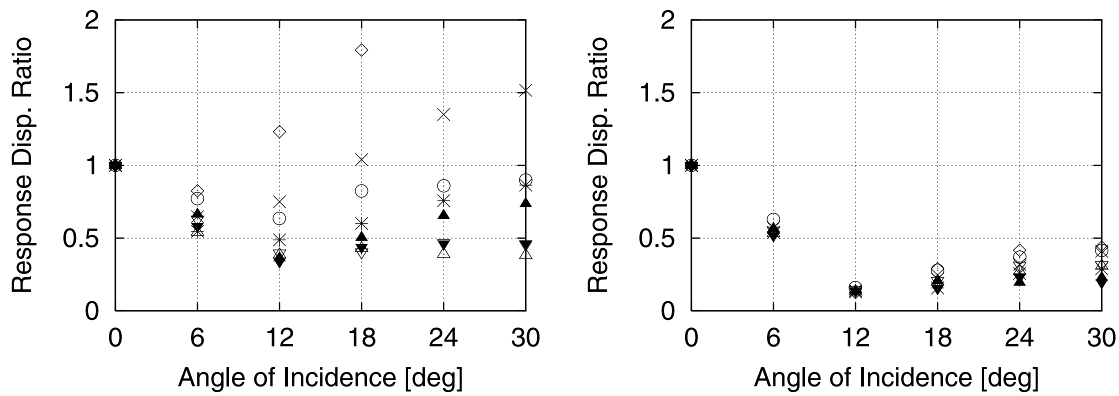
The elastic response analyses are conducted using the Wilson's θ method. As the input ground motions El Centro (Imperial valley earthquake, 1940), Hachinohe (Tokachioki earthquake, 1968), JMA Kobe (Hyogoken nanbu earthquake, 1995) and Taft (California earthquake, 1952) are used. Tab.3 shows the legends used in Fig.11-18.

In Fig.11-18 earthquake responses are estimated by "Response Displacement Ratio" which is the ratio of response displacement by phase difference inputs to one by uniform inputs (that is θ=0°).

Fig.11 and 12 show the effects of phase difference. ℓ =192m and k_f / k_s =8 are assumed. According to Fig.5 when the phase difference exists, amplification factor of 1st mode is relatively small and 2nd or above mode vibration affects to responses. So scatter of response displacement ratio becomes large at the left side lumped mass in the case of V_s =100m/s. According to Fig.6 as amplification factor of 1st mode is relatively large, scatter of response displacement ratio in Fig.12 is smaller. In the case of the central lumped mass, scatter is small and the reduction effects by phase difference are recognized.

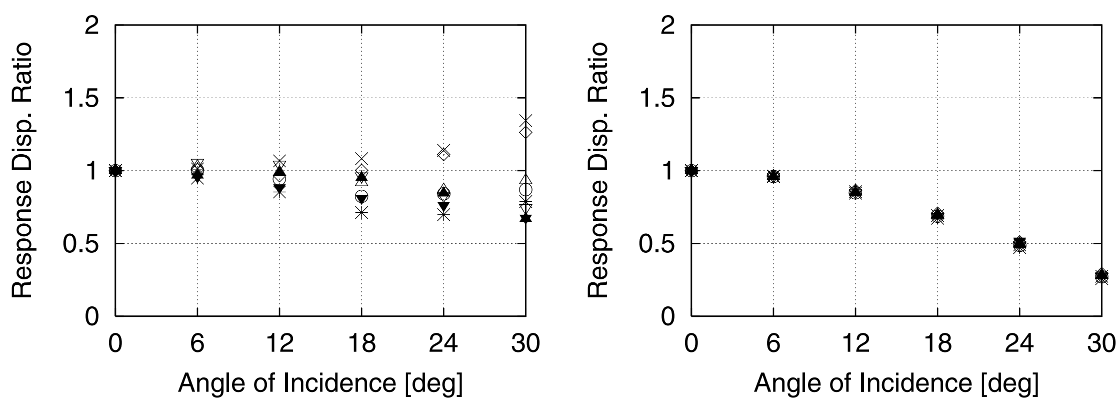
Tab.3 Legends of Fig.11-18

El Centro NS	○
El Centro EW	◇
Hachinohe NS	✳
Hachinohe EW	×
JMA Kobe NS	▽
JMA Kobe EW	△
Taft NS	▲
Taft EW	▼



left side lumped mass　　central lumped mass

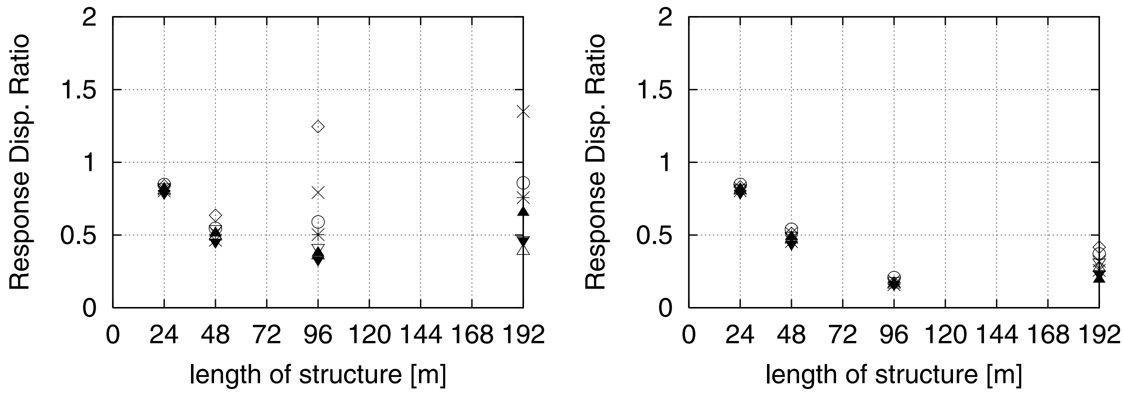
Fig.11 Effects of phase difference (V_s =100m/s)



left side lumped mass　　central lumped mass

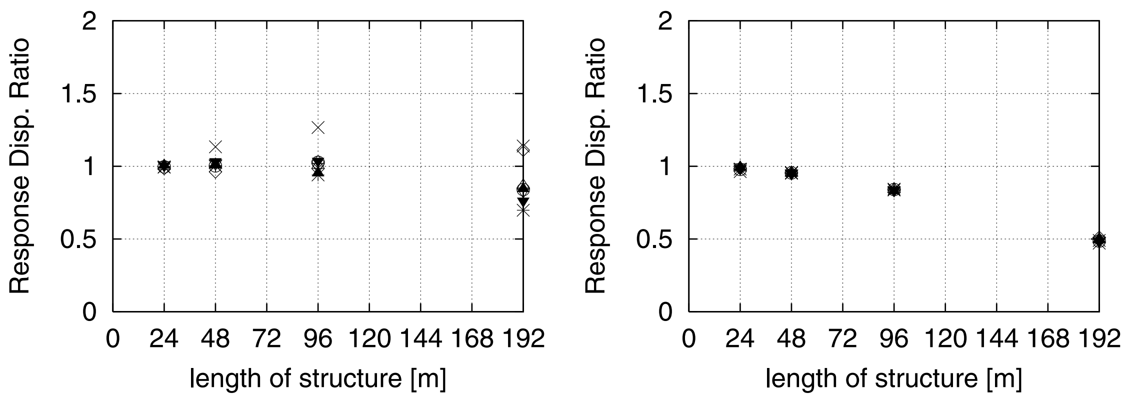
Fig.12 Effects of phase difference (V_s =400m/s)

Fig.13 and 14 show the effects of overall length of a structure. $k_f\ /\ k_s$=8 and θ=24° are assumed. General tendency is same as the effects of phase difference. When length of a structure is longer, value and scatter of response displacement ratio becomes smaller. In particular, according to Fig.7, amplification factor of 2nd and above mode is much smaller at the left side lumped mass, so the scatter of response displacement ratio becomes much smaller(left of Fig.13).



left side lumped mass central lumped mass

Fig.13 Effects of overall length of a structure (V_s=100m/s)



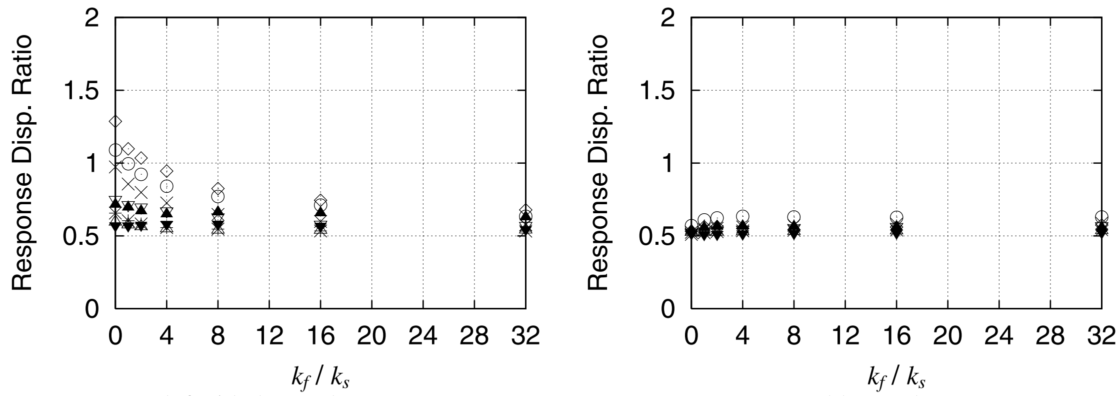
left side lumped mass central lumped mass

Fig.14 Effects of overall length of a structure (V_s=400m/s)

Fig.15to 18 show the effects of stiffness of foundation slabs. ℓ=192m and θ=24° are assumed.

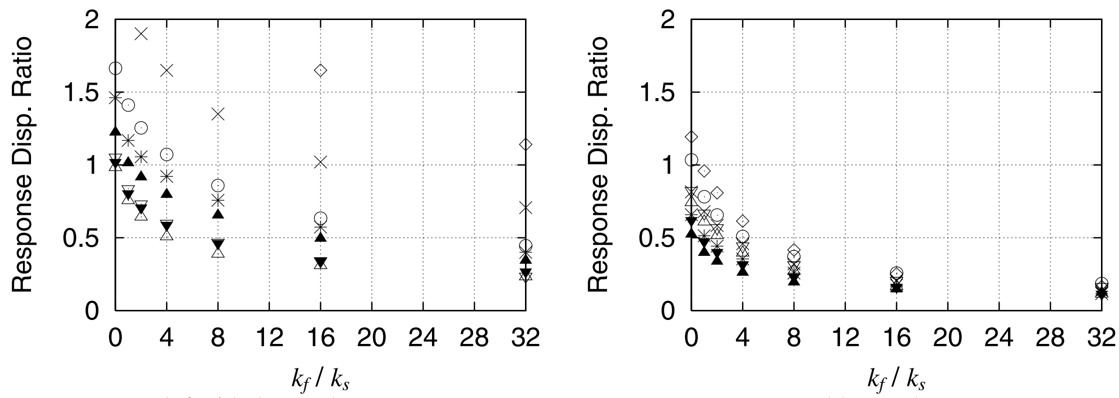
In the case of V_s=100m/s, generally the values of response displacement ratio are tend to be small, and the scatters are large at the left side lumped mass. When θ=24°, the effects of stiffness of foundation slabs are large. The scatter is large but reduction tendency is also large. This is because of the complex tendency shown in Fig.9.

In the case of V_s=400m/s, the effects of stiffness of foundation slabs is very small. The scatter and the effect of θ are also small. In Fig.18 at the central lumped mass almost all the values are 0.5. This is because that in Fig.11 amplification factors are generally small in the all range.



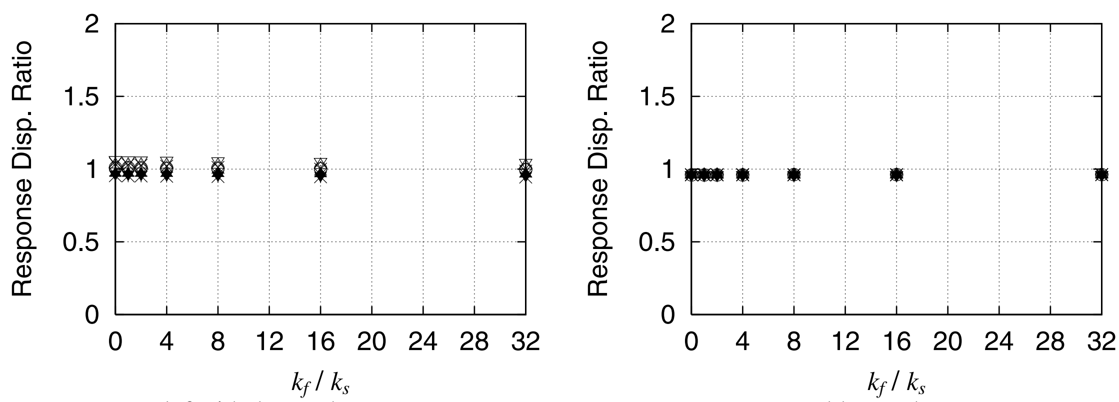
left side lumped mass central lumped mass

Fig.15 Effects of stiffness of foundation slabs (V_s=100m/s, θ=6°)



left side lumped mass central lumped mass

Fig.16 Effects of stiffness of foundation slabs (V_s=100m/s, θ=24°)



left side lumped mass central lumped mass

Fig.17 Effects of stiffness of foundation slabs (V_s=400m/s, θ=6°)

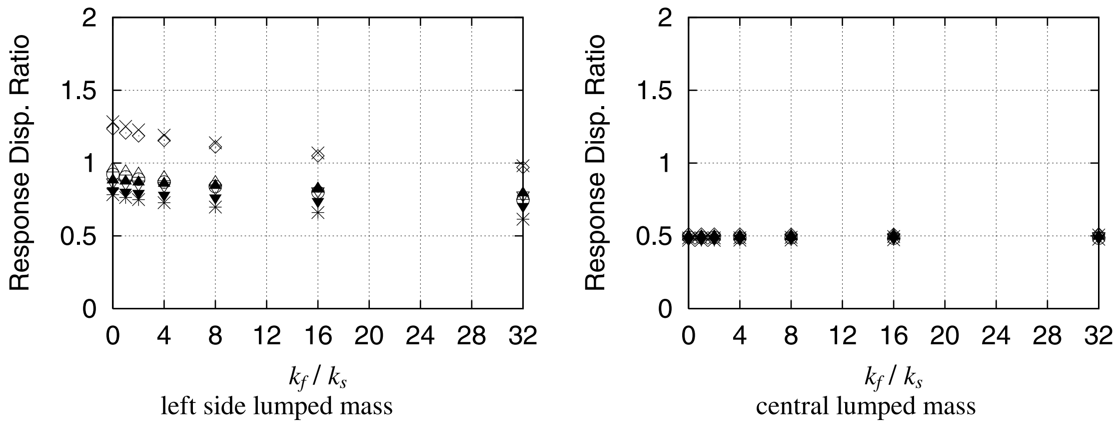


Fig.18 Effects of stiffness of foundation slabs (V_s=400m/s, θ=24°)

CONCLUSIONS

In order to examine dynamic behaviors of structures due to ground motions considering phase differences, the elastic response analyses were conducted using the multi-degree-of-freedom system model which consist of the unit models serially connected by the floor slab spring and the foundation slab spring. SH waves inputted obliquely from perpendicular downward were considered as input ground motions.

The results are summarized like the following. The response of the structure due to the input ground motion considering phase differences becomes smaller than that due to the uniformly incident input ground motion, since the translation component decreases. However, there is a case in which the response increases on the outermost part of the structure by the effect of higher mode of the structure system, and its tendencies are remarkable for the case in which the structural overall length is long, for the case in which the rigidity of the foundation slab is not high, and for the case in which the ground is softer.

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