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# SEISMIC ANALYSIS OF MULTI-SPAN CONTINUOUS GIRDER BRIDGE WITH EMPHASIS ON SOIL-FOUNDATION-SUPERSTRUCTURE INTERACTION

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In this paper the application of the computer program SUBSSIP-2D (Substructure method for Soil-Structure Interaction Problem: 2-Dimentional Analysis) is made for the seismic analysis of a three-span continuous girder bridge on multiple foundations of deep grouped piles and wall types. The SUBSSIP-2D is developed based on the substructure method, which first evaluates the subgrade impedance at the specified foundation nodes and the effective seismic input motions to them, and then establishes the inertial coupling between substructure and superstructure. The detail comparison is presented among solutions from various modelings and methods of analyses to derive the useful design considerations for bridges dealt with herein.

## 1. INTRODUCTION

Structural response during earthquake motions is greatly affected by the soil condition for certain cases as to show quite different response features from when it is fixed at its base. Free field analysis exhibits that the soil amplification for body waves is most significant at the soft soil near surface so that the control point for seismic analysis is usually placed at the base rock level (rigid base) or at the level below which a uniform soil extends to infinite (halfspace base). Thus, the seismic analysis is concentrated on the near surface region.

The finite element approach is widely used for the soil-structure interaction analysis for its adaptability that can accommodate the complex boundary geometry. The 2-dimensional, axisymmetric or 3-dimensional modeling is taken depending on the structure for analysis. Exclusively, uniform seismic waves are imposed at the base of the model[1]. The earthquake observation is, however, detecting substantial traveling nature of seismic waves. This finding, coupled with the recent trend of constructing of large-scale extended structures like continuous span bridges, reveals the limitation of such a uniform seismic input. The traveling nature is becoming one of the important factors to be incorporated in the seismic analysis of those structures.

The direct solution method[1] for the complete soil-structure interaction system is hardly preferable since the total degrees of freedom become tremendously large for the extended large-scale structures or costly if it is excuted. The simplification of the soil-structure interaction problem is often taken. One such an approch,[2] which is common practice in the design process in order to make use of the conventional computer program, is to separate the solution process into two phases; the first phase is the analysis for the soil-foundation system (substructure) with reduced superstructural model, and the second phase is

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the superstructural analysis for the foundation motion from the first phase as input in return. In this method the rigorous interaction is not considered between substructure and superstructure.

The auther has proposed a new formulation[3] using the Ritz vector method for the soil-structure interaction analysis to meet the above claim and called it dynamic substructure method. Further, this concept is implemented into the computer program SUBSSIP-2D.[4)~6)]

The SUBSSIP-2D is characterized by the following features:

1) The 2-dimensional seismic waves may be simulated as body waves and/or surface waves.

2) The kinematic interaction may be evaluated between soil and foundation to derive the soil impedance and the effective seismic driving force at specific nodes of foundation.

3) The rigorous formulation for a complete system of substructure and superstructure may be obtained and an efficient reduction of degrees of freedom may be adopted for the complete system response analysis.

4) For the soil modeling the energy radiation from foundation into the far field may be accounted for by introducing appropriate side and bottom boundaries.

The present paper first reviews briefly the SUBSSIP-2D and then reports, as illustrative example by this, on the seismic analysis results of a multi-span continuous girder bridge on pile and wall foundations (Fig.1).

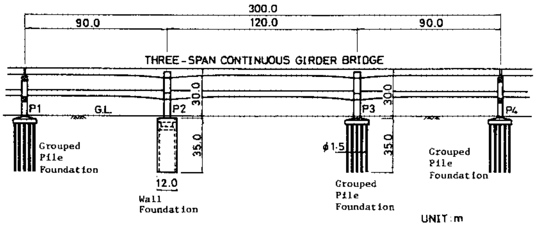



Fig. 1 General View of Structure.

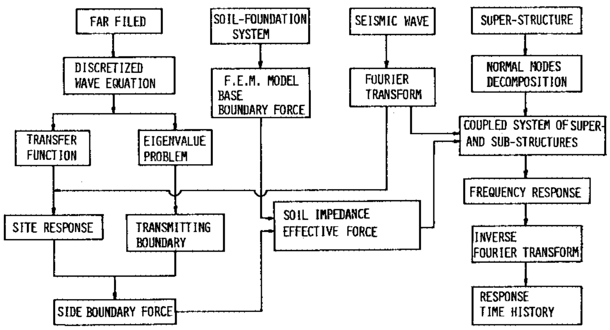



Fig. 2 Analysis Flow for SUBSSIP-2D.

## 2. COMPUTER PROGRAM : SUBSSIP-2D

The contents comprise the four processes: (i) Site response analysis, (ii) Substructure analysis, (iii) Superstructure analysis, and (iv) Interaction analysis for the complete system. Fig.2 shows the flow of analysis.

( i ) Site response analysis

The finite element technique is used such that the wave amplitudes vary linearly between adjacent nodes along the depth within surface layers. In order to account for the waves radiating into the half-space base the hybrid approach by Chen et al.[7] is adopted that connects the former discrete solution with the latter continuous solution at the top of the base. For the body wave input, the control point is set on the base top level. The body wave of any incident angle f can propagate in the horizontal direction (x-dierction) by the wave number

$$k=\frac{\omega \sin \mathrm{f}}{V_s} \quad \text{for shear wave} \quad \text{or} \quad k=\frac{\omega \sin \mathrm{f}}{V_p} \quad \text{for pressure wave} \qquad (1)$$

in which $\omega$=frequency, $V_s$=shear wave velocity, $V_p$=pressure wave velocity in the base rock. Hence, the soil displacement is expressed as

$$\boldsymbol{u}_j^*(x,z_j)=\boldsymbol{u}_j^*(z_j)e^{i(\omega t-kx)}, \quad j=1,\cdots,N \qquad (2)$$

In simulating surface waves, on the other hand, a control point may be chosen at the surface. The half-space base is equivalently replaced by additional layers whose depth varies with frequency. The

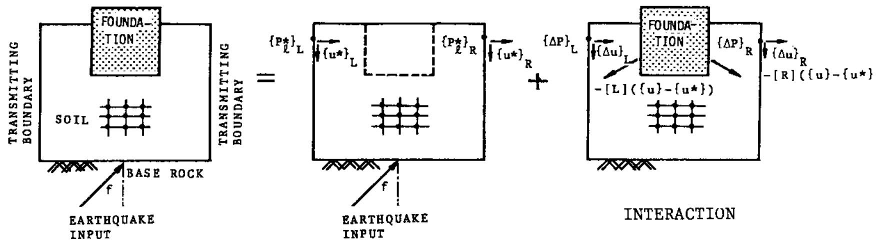



Fig. 3 Superposition for Wave Field.

application is referred to Ref 7).

(ii) Soil-foundation (Substructure) analysis

The near field that encompasses the foundation and the soil in its vicinity is modeled by the 2-dimensional finite elements. In order to account for the wave radiation from foundation into far field, the transmitting boundary[8] is introduced at side boundaries.

The governing equation for this substructure is then expressed, in view of Fig. 3, as

$$(\boldsymbol{K}_{\text{sub}}-\omega^2 \boldsymbol{M}_{\text{sub}})\boldsymbol{u}_{\text{sub}}=\boldsymbol{P}_L+\boldsymbol{P}_R+\boldsymbol{P}_{\text{sup}} \quad (3)$$

in which $\boldsymbol{K}_{\text{sub}}$ and $\boldsymbol{M}_{\text{sub}}$ denote, respectively, the stiffness and mass matrices; $\boldsymbol{P}_L$ and $\boldsymbol{P}_R$, the force vectors acting at the left and right side boundaries, are computed by

$$\boldsymbol{P}_{L,R}=\pm[\boldsymbol{K}_l \mid \boldsymbol{K}_b]\begin{Bmatrix} \boldsymbol{u}_l^* \\ \boldsymbol{u}_b^* \end{Bmatrix}_{L,R}-\boldsymbol{R}_{L,R}(\boldsymbol{u}_{L,R}-\boldsymbol{u}_{lL,R}^*) \quad (4)$$

in which $[\boldsymbol{K}_l \mid \boldsymbol{K}_b]$ signifies the expanded 1-dimensional soil stiffness, $\boldsymbol{u}^*$ is the corresponding response vector, and the suffix attached indicates that b is for base top level nodes and $l$ for other nodes in layered soils. $\boldsymbol{u}_{L,R}$ is the displacements at side boundary and part of the whole substructure displacements $\boldsymbol{u}_{\text{sub}}$. The maxtrix $\boldsymbol{R}_{L,R}$ is the transmitting bounary element. The forces vector $\boldsymbol{P}_{\text{sup}}$ is the direct force acting on the foundation from the superstructure.

The main purpose of this process is to prepare the impedance and the effective seismic input which are used in the suceeding process to couple with the superstructural characteristics for the complete system analysis. The alternative solution methods, either one-step solution or the dynamic substructuring, are available. The former approach evaluates the impedance functions and the effective forces at the junction nodes with the superstructure through the condensation process in Eq.(3). The substructuring method, on the other hand, specify the intercommon nodes between soil and embedded foundation. Two different specification is possible, by taking them at the soil-structure interface or virtually within the embeded portion of foundation. These are respectively refered to as the interface modeling and as the interbody modeling[3]. Either way of analyses results in the governing equation of, for the soil-foundation system

$$X_{\text{sub}}\,\boldsymbol{u}_{\text{sub}}^{I}=\boldsymbol{P}_{\text{sub}}^{I}+\boldsymbol{P}_{\text{sub}}^{0} \quad (5)$$

in which $X_{\text{sub}}$ defines the corresponding subgrade impedance matrix, $\boldsymbol{P}_{\text{sub}}^{0}$ the effective seismic input vector and $\boldsymbol{P}_{\text{sub}}^{I}$ the internal force vector with the superstructure.

(iii) Superstructural analysis

The normal modes decomposition is presumed for this part of the structure since the damping effect is generally small as being several percent at most of the critical values. Besids, most of the engineers are so familiar with this solution method. Hence

$$\boldsymbol{u}_{\text{sup}}^{\text{d}}=\boldsymbol{\Phi}_{\text{sup}}\,\boldsymbol{q}_{\text{sup}} \quad (6)$$

in which $\boldsymbol{\Phi}_{\text{sup}}$ is the modal matrix and $\boldsymbol{q}_{\text{sup}}$ is the modal response vector.

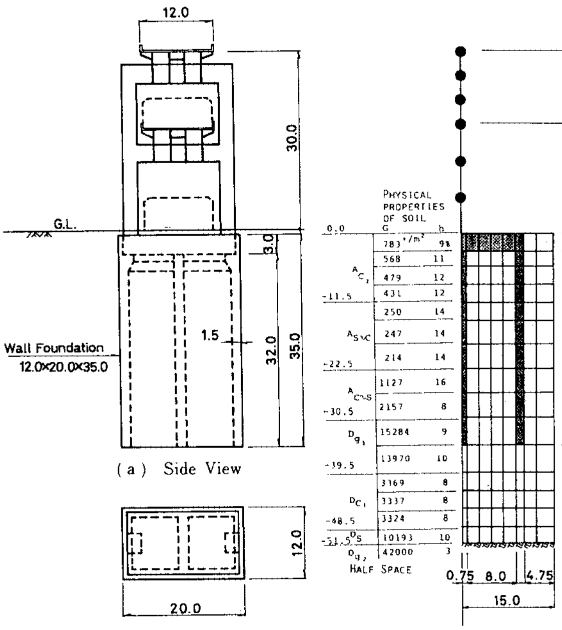
(iv) Interaction analysis

The coupling between substructure and superstructure is established by claiming the displacement compatibility and force equilibrium at their junction nodes. The superstructural response $\boldsymbol{u}_{\text{sup}}$ is now accounted for as the sum of the dynamic one $u^{d}_{sup}$ due to the inertial coupling at the fixed base state and the quasi-static one $u^{l}_{sup}$ due to releasing the constraint condition at the superstructural base according to the foundation degrees of freedom, i.e.,

$$u_{sup}=u^{d}_{sup}+\beta_{sup}\,u^{l}_{sub} \qquad (7)$$

in which $\beta_{sup}$ defines the displacement influence matrix to be obtained from the static analysis. The expression of Eq.(7) implies that the present solution method is a Ritz vector method.[9] An efficient reduction of degrees of freedom for response analysis is possible in Eq. (6) by truncating the higher small contributing superstructural modes in view of the frequency contents of the input seismic motions.

## 3. SOIL-FOUNDATION-PIER SYSTEM




( a ) Side View

( b ) Plan View

( c ) FEM Model

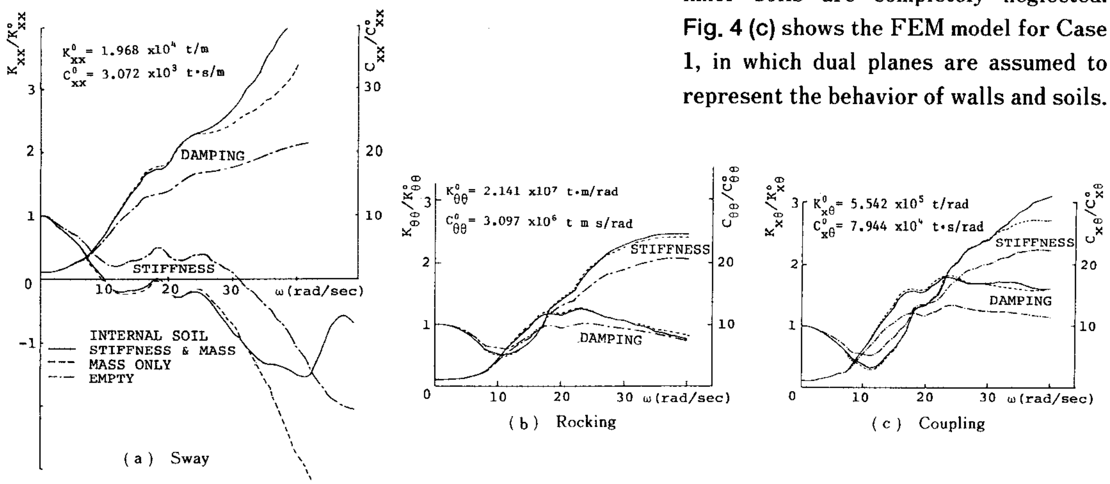
Fig. 4 Wall Foundation.

First, the dynamic characteristics are investigated for the isolated soil-foundation pier systems in the direction perpendicular to the bridge axis.

( i ) Wall Foundation

This foundation, located at P2 in the complete system in Fig. 1, is popular in Japan for its convenience of construction. This foundation, as shown in Fig. 4 (a) and (b) being different from the caisson foundation, has undisturbed original soils inside walls which is constructed at site, and its seismic design provision is yet to be established. The interest for investigation is thus placed on the behavior of inner soils.

Herein, three different models are considered: Case 1- both the mass and stiffness of inner soils are taken into account, Case 2-only the inner soils mass effect is considered, and Case 3-inner soils are completely neglected. Fig. 4 (c) shows the FEM model for Case 1, in which dual planes are assumed to represent the behavior of walls and soils.




( a ) Sway

( b ) Rocking

( c ) Coupling

Fig. 5 Impedance Functions of Wall Foundation.

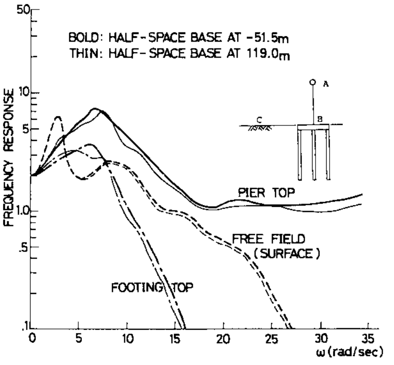



Fig. 6 Frequency Responses of Wall Foundation.

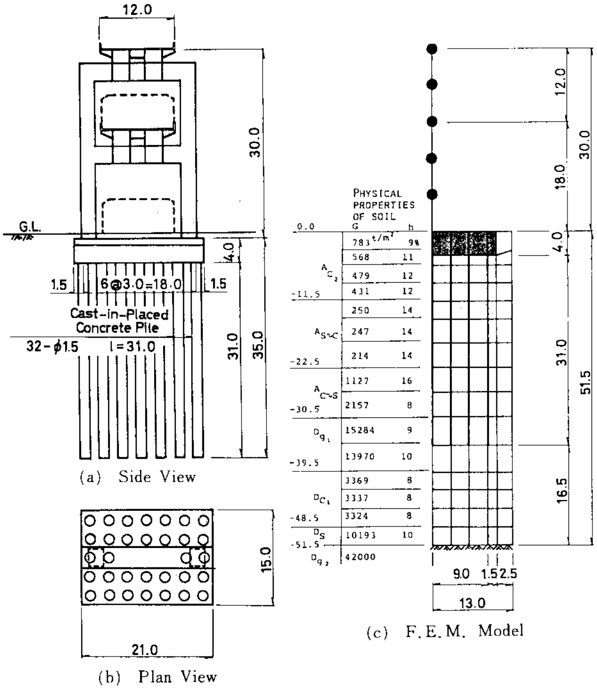



Fig. 7 Grouped Pile Foundation.

They are connected by soil shear rigidity and bonded at wall faces. The top mass at the pier is associated with the girder mass.

In Fig. 5 are shown the impedance functions of the soil and foundation system evaluated at the footing top. It is noted that the inner soils move almost in phase with the foundation and indicate primarily the inertial effect rather than the stiffness effect against wall motion. Fig. 6 depicts the frequency responses at the footing top and the pier top as well as at the free field surface in order to see the soil-fundation interaction phenomenon. A predominant peak response appears, due to the interaction, at the higher frequency than the fundamental free field frequency. This interaction mode is dominated by the foundation rocking motion, judging from the amplification from its top to the pier top in view of the rigidity of the pier. In Fig. 6, the placing level of the half-space base is also checked by putting additional layers for half-space portion. The deeper level is determined on the basis that the associated Rayleigh wave amplitude at the predominant interaction frequency is neglegibly small. The previous model is noted to be adequate.

( ii ) Grouped Pile Foundation

The superstructure is supported by this type of fundation at P1, P3 and P4 foundations. The grouped pile foundation in Fig. 7 (a),(b) is designed as statically equivalent to the foregoing wall foundation on the design code basis. Fig. 7 (c) is the FEM model for analysis in which beam elements are used for piles.

In Fig. 8 are shown the impedance functions of the soil-foundation system evaluated at the footing top. When they are compared with those of the wall foundation, the grouped pile foundation impedances are indicative of the more flexible

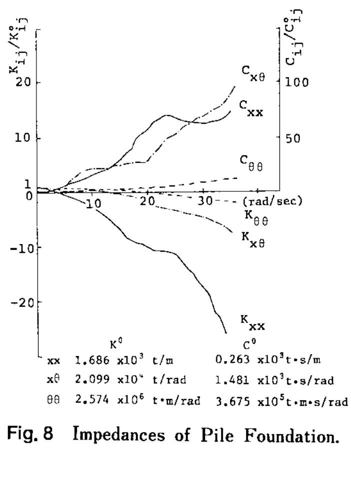



Fig. 8 Impedances of Pile Foundation.

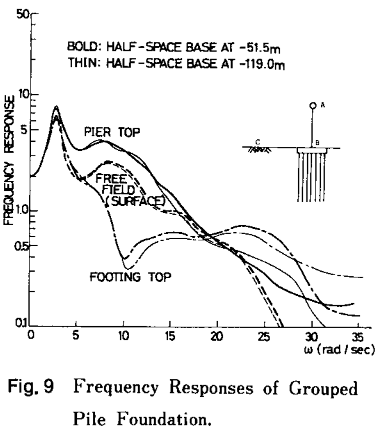



Fig. 9 Frequency Responses of Grouped Pile Foundation.

nature. Fig. 9 is the frequency responses at the footing top and pier top levels as well as that of the free field surface. It is noted that the grouped pile foundation is much affected by the free field vibration modes in the low frequency range and the inertial interaction is negligibly small. The check of the level of the half-space is made in Fig. 9 as likewise in the wall foundation, from which it may be judged that the setting in Fig. 7 (c) is adequate.

## 4. SUPERSTRUCTURE

The girder and pier portion in Fig. 1 is defined as superstructure herein. The lumped mass modeling is taken as in Fig. 10. The results of the normal modes analysis are shown in Fig. 11 only for those of significant participation factors. Interesting to note is that the predominantly contributing modes differ as the section considered. For the response analysis in what follows, the modal damping of 2 percent of critical values is assumed.

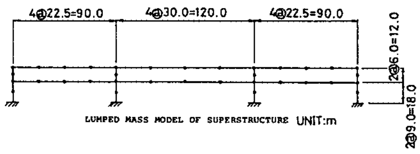



Fig. 10 Lumped Mass Model for Superstructure.

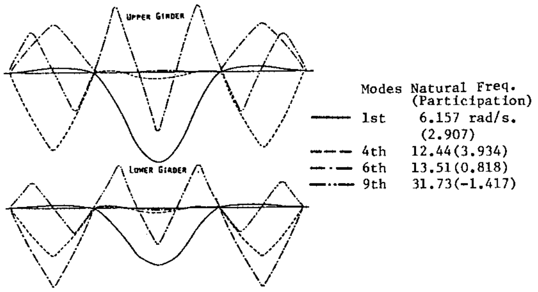



Fig. 11 Normal Modes of Superstructure.

## 5. COMPLETE (SOIL-FOUNDATION-SUPERSTRUCTURE) SYSTEM

Since the bridge for the present analysis is an extended structure on multiple different types of foundations, the interest is placed on the modeling as well as on the method of analysis. The following case studies are carried out: ( i ) Rigorous interaction analysis for a uniform seismic input at the base rock level, ( ii ) Rigorous interaction analysis for a progressive seismic input at the base rock level as SH wave along the bridge axis from left to right, (iii) Approximate response by using the free field surface response as input for the superstructure with soil-foundation impedance at its base, (iv) Analysis with fixed base assumption for the superstructure, and ( v ) Simplified interaction analysis as stated in INTRODUCTION. For the analyses ( i ) and ( ii ), the impedance functions of the respective foundation and the corresponding effective seismic driving forces are used.

In order to grasp the dynamic characteristics of an integrated system of substructure and superstructure, the frequency response function is first investigated. It is noted that the pier behavior is characterized by its respective foundation as in the case of the foregoing isolated pier-foundation system analysis. Namely, the pier on the pile foundation is much affected by the soil vibration modes (Fig. 12 (a), (c), (d)), indicating predominant frequency contents of the soil fundamental frequency. The approximate analysis with use of the soil surface response as input motion results in a reasonable response estimate, although it lacks the rigorous kinematic interaction between soil and foundation. The traveling seismic input along the bridge axis shows a smaller response at the first pier P1 than the corresponding value in the uniform input situation. However, it gives a growing response along the wave propagation direction and then at the last pier P4 a greater response than that in the uniform input case. The approximate interaction analysis results in a reasonable response estimate at the important frequency range from the seismic analysis. At the pier top on the wall foundation (Fig. 12 (b)), on the other hand, the inertial interaction between soil and structure is significant, so that the approximate input with use of the soil surface response leads a poor response estimate. The approximate interaction analysis yields a smaller response below the predominant frequency due to the inertial interaction, while a greater response beyond it.

The girder response is investigated in connection with the soil-structure interaction. The fact that the dominating vibration modes differ at the respective span, is clearly noted in the frequency domain. Furthermore, the interaction effect is appreciable. At the center span where the superstructural first mode contribution is predominant, it appers in the high frequency range than this frequency, diminishing the

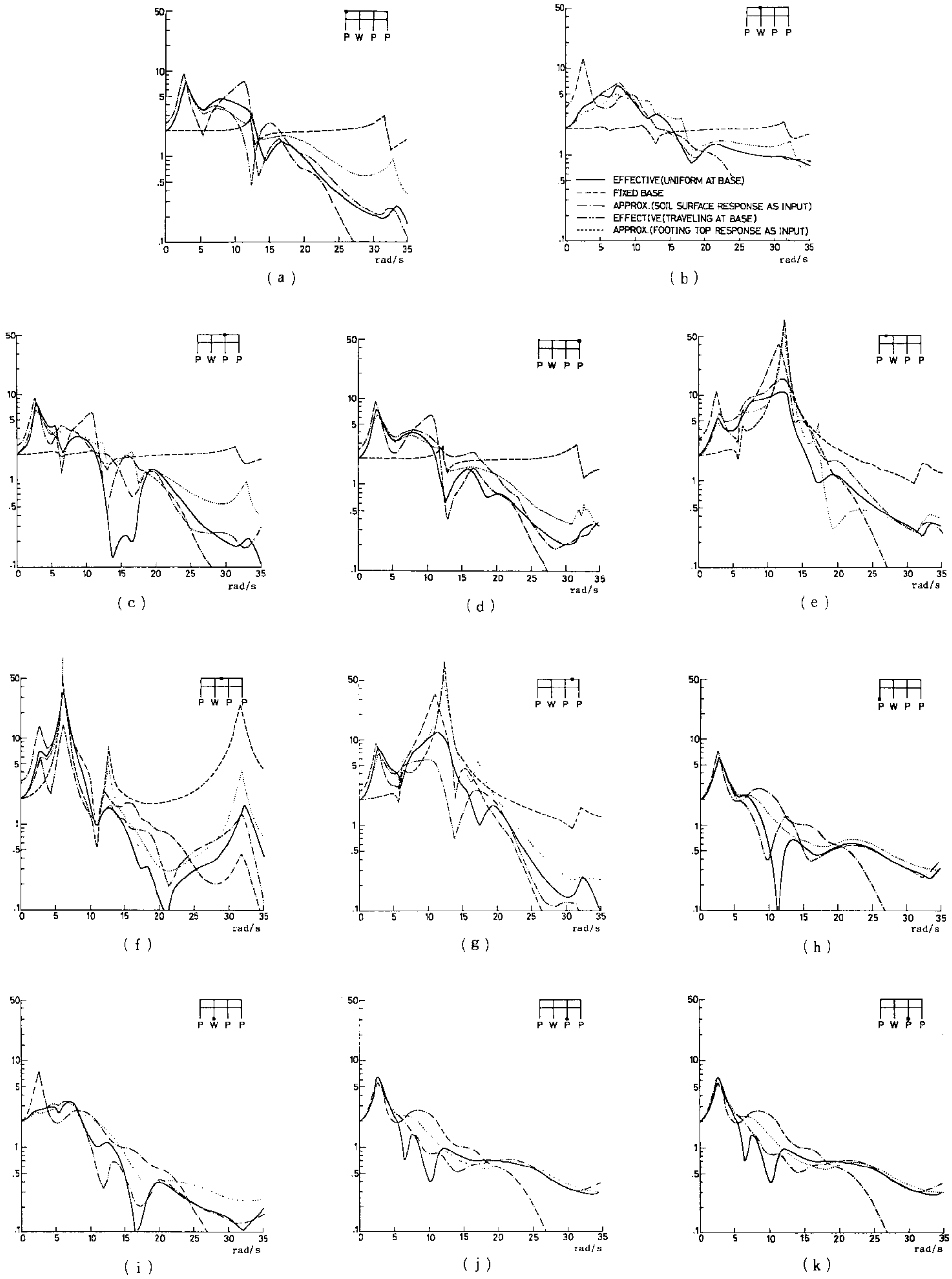



Fig.12 Frequency Responses of Complete System.

response significantly (Fig. 12 (f)). At the side spans where the superstructual 4th mode is predominant, the response around this frequency is directly affected to such an extent that the response characteristic is greatly changed (Fig. 12 (e), (g)). The use of the soil surface response as input gives an overestimate to the rigorous interaction solution. The traveling input as SH wave results in a larger response at the left side span while a smaller response at the right side span when compard with the response for the uniform input case. This phenomenon is due to the fact, in view of Fig. 11, that the 4th and 6th superstructural modes are amplified with the phase difference of input motion. The approximate interaction method yields a poor response estimate, retaining the superstructural modes with imposed small amount of damping at this portion.

The Fig. 13 shows the maximum acceleration response for the earthquake TAFT 1952, N21W component as an input. The larger maximum response is attained at the side spans rather than at the central span. The soil-structure interaction makes the response reduced. This trend is more appreciable at the span centers than at the pier tops. The traveling input yields a smaller response at the first pier P1 and the growing response along the wave propagation direction and then at the last peir P4 the response is larger than that for the uniform input. As for the girder response the traveling input results in a larger response at the left side span but at the central and right side spans smaller responses than those for the uniform input. This is reasoned such that when the pier response becomes large due to the phase difference of input motions, the response at midspans is decreased. The more quantitative explanation is made based on the information of input and structural dynamic characteristics.[10)~11)] The use of the soil surface response as input and the approximate interaction analysis, as are indicated from the frequncy response analysis, give an overestimation for response.

The maximum internal froces are most important from the structural design point of veiw. Those at the pier foot represent the total seismic forces. Fig. 14 indicates that the soil-structure interaction reduces them from when the structure is rigidly supported state. The approximate interaction analysis gives an overestimation for these; and in the worst the estimation is two times larger of the rigorous solution at the pier P4. The traveling input leads a quite different response features with the dynamic characteristic of the present structure. The approximate input with use of the soil surface response results in an overestimation for the response.

## 5. CONCLUSIONS

In this paper, a brief remark has been given for the soil-structural interaction analysis system SUBSSIP-2D, which is developed based on the substructuring method, and the discussion is made on the numerical results for an illustrative example of a three-span continuous bridge of mixed foundations of

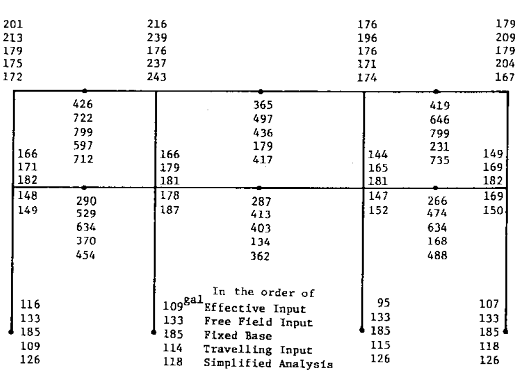



Fig.13 Maximum Response Acceleration
Input: TAFT 1952, N21W Max.Acc.100 gal.

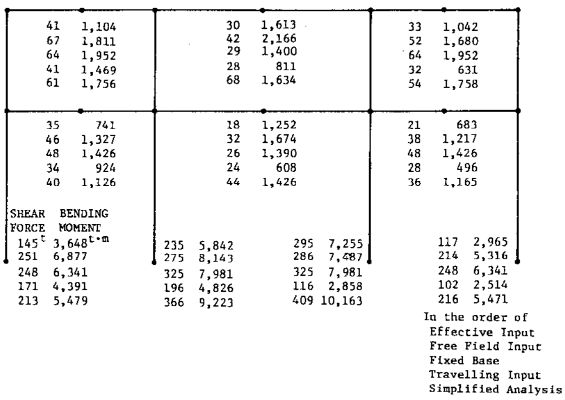



Fig.14 Maximum Internal Forces
Input: TAFT 1952, N21W Max.Acc.100 gal.

three grouped-pile foundations and a wall type foundation.

The following useful information is derived through the analysis for a bridge structure dealt with herein:

(1) The wall foundation in present study primarly tends to show an inertial interaction mode with the surrounding soils like a massive rigid caission foundation.

(2) The grouped-pile foundation herein tends to obey the soil vibration mode in the important low frequency range from the seismic analysis.

(3) The soil surface response may be used as an acceptable input motion for a flexible pile foundation but may not be appropriate for a foundation which has a strong kinematic interaction with the surrounding soils.

(4) The approximate interaction analysis, which devides the complete analysis as substructure of soil-foundation system with a simple model attached to represent the superstructural inertial force feedback, for the convenience sake in order to use conventional computer programs, is not recommended since it lacks the rigorous inertial interaction and tends to yield overestimated response.

(5) The 2-dimensional propagating nature of seismic waves is important as to change the response features from those for a uniform input situation at the base rock.

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REFERENCES

1) Lysmer, J., Udaka, T., Tsai, C.F. and Seed, H.B. : FLUSH, A Computer Program For Approximate 3-D Analysis of Soil-Structure Interaction Problem, EERC Report No.75-30, Univ. of Calif., Berkeley, 1975.

2) JSCE : Technical Report on the Seismic Analysis and Soils of Honshuu-Shikoku Straight Bridges, 1979.

3) Takemiya, H. : Analysis of Soil-Foundation-Superstructure Interaction During Earthquake Motions- Dynamic Substructure Method, Tsuchi-to Kiso (Soils and Foundation), Vol.29, No.9, pp.27～34, 1981.

4) Takemiya, H., Shimada, T., Tatsumi, M. and Katayama, M. : Seismic Analysis of Multi-span Continuous Girder Bridge with Emphasis on Soil-Structure Interaction, The 6th Japan Earthquake Engineering Symposium, pp.1193～1200, 1982.

5) Takemiya, H. : Seismic Analysis System for Soil-Structure Interaction Problem by Dynamic Substructuring Method : SUBSSIP-2D, The 17th Symposium on Earthquake Engineering Research, pp.175～178, 1983.

6) Takemiya, H. : Seismic Response Characteristics of a Multispan Continuous Girder Bridge, The Bridge and Foundation Engineering, Vol.17, No.10, pp.45～52, 1983.

7) Chen, J.C., Lysmer, J. and Seed, H.B. : Analysis of Local Variation in Free Fields Seismic Ground Motion, EERC Report, No.81-03 Univ. of Calif., Berkeley, 1981.

8) Lysmer, J. and Drake, L.A. : A Finite Element Method for Seismology, Methods in Computational Physics, Vol.11, Academic Press, pp.181～216, 1972.

9) Hurty, W.C., Collings, J.D. and Hart, G.C. : Dynamic Analysis of Large Structures by Modal Synthesis Technique, Computers and Structures, Vol.1, pp.553～563, 1971.

10) Takemiya, H. and Kai, S. : Seismic Analysis of a Multispan Continuous Bridge on Deep Pile Foundations, Procs. of Japan Society of Civil Engineers, No.332, pp.1～10, 1983.

11) Yamada, Y., Takemiya, H., Kawano, K. and Hirano, A. : Earthquake Response Analysis of High-Elevated Multi-span Continuous Bridge on Dynamic Soil-Structure Interaction, Procs. of Japan Society of Civil Engineers, No.328, pp.1～10, 1982.

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