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SEISMIC PERFORMANCE EVALUATIONS OF A BRIDGE STRUCTURE WITH NONLINEAR DYNAMIC INTERACTION

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SUMMARY

For the reliable design of structures subjected to severe seismic forces, it is very important to make the nonlinear dynamic response characteristics clearly. Nonlinear seismic responses analyses for framed structures with nonlinear soil-structure interaction are carried out in the present study. The strength demand spectra play important roles on the nonlinear seismic response evaluation from the design points of views. The applicability of the strength demand spectra to a bridge structure system with a multi-degree-of-freedom (MDOF) is examined by performing the nonlinear seismic response analyses. It is suggested that since the nonlinear characteristics plays significant roles on the seismic response evaluation, reliable evaluations of the seismic responses can be carried out using the total system with interaction. Damage assessment due to drift displacement can be approximately obtained from the examination of the total system.

INTRODUCTION

In order to enhance the reliability of structures due to seismic motions, the nonlinear seismic response plays important roles in the design of the structures. Seismic response analyses for framed structures with soil structure interaction, which can be represented with the MDOF system, are carried out using a strength demand spectra and the incremental method. Since the seismic motion has stochastic properties, deterministic evaluations of the responses cannot always account for the exact response quantities (Wen[2]). Taking into account for the randomness of the seismic motions, the increase of the strength of the structure does not always give the efficient means for the seismic design of the structure. If some damages on the structure can be allowed to the responses against severe seismic motions, it is very available to perform the seismic design of the structure such as described by the performance based design method (Iemura et al [1]). The performance based design approach will help in developing more effective methods on seismic resistant design of civil engineering structures. The performance-based design is recommended in these days and it is essential to make the response characteristics of the structures clear in the nonlinear region. Since the dynamic response of the total system closely depends upon the foundation and the soil conditions, it is important to take into accounts for the soil structure interaction. In order to carry out the performance based design on the total system, it is necessary to find out soil structure interaction effects for the reliable seismic design.

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Thus it is essential to deal with the total system consisted of superstructure as well as soil foundation system and to estimate nonlinear response characteristics of this total system (Toki et al [3]). Moreover, damage evaluation of the structure subjected to seismic forces plays important roles on the reliable design of the structure. There are many researches on the damage evaluations for seismic forces (Fajfar[4]). It is important to make the damage assessment clear for the MDOF structure system with soil structure interaction.

The nonlinear dynamic response analysis is therefore carried out for framed structures with soil-foundation subjected to severe seismic motion in the present study. Applicability of the strength demand spectra with respect to the MDOF system subjected to severe seismic motion is also examined. Damage evaluation of the soil structure interaction system is carried out using a drift displacement of the structure.

FORMULATION

A bridge structure supported by soil foundation is known to have very complicate characteristics on dynamic response evaluations. The superstructure with piers is represented with the framed structure which is modeled by the finite element method in the present study. If the structure is subjected to severe seismic forces beyond the required design level, the response certainly comes out nonlinearity. The nonlinear characteristics of the superstructure can be generally represented with the nonlinear relation between the bending moment and curvature.

The framed structure generally corresponds to the MDOF system. The nonlinear dynamic response analysis is carried out using the analytical model as shown in Fig.2. The superstructure is formulated with the finite element method (FEM) model by the beam element and the soil pile foundation is modeled by the sway-rocking model (SR model). The nonlinear characteristic of the superstructure is expressed with the typical bilinear model using $M-\phi$ (the bending moment curvature) relation. The governing equation of motion of the superstructure is expressed as follows.

$$\begin{bmatrix} [M_{aa}] & [M_{ab}] \\ [M_{ba}] & [M_{bb}] \end{bmatrix} \begin{Bmatrix} \{\ddot{u}_a\} \\ \{\ddot{u}_b\} \end{Bmatrix} + \begin{bmatrix} [C_{aa}] & [C_{ab}] \\ [C_{ba}] & [C_{bb}] \end{bmatrix} \begin{Bmatrix} \{\dot{u}_a\} \\ \{\dot{u}_b\} \end{Bmatrix} + \begin{bmatrix} [K_{aa}] & [K_{ab}] \\ [K_{ba}] & [K_{bb}] \end{bmatrix} \begin{Bmatrix} \{u_a\} \\ \{u_b\} \end{Bmatrix} = \begin{Bmatrix} \{F_a\} \\ \{F_b\} \end{Bmatrix} \tag{1}$$

If he equation of motion of the soil pile foundation is expressed with the SR model which is modeled with the stiffness for the sway and the rocking motion (Kawano et al [5]), it is very available to combine the superstructure with the soil foundation system by means of the substructure method. The equation of motion of a soil pile foundation system is expressed as follows:

$$\lfloor M_p \rfloor \{\ddot{x}_p\} + \lfloor C_p \rfloor \{\dot{x}_p\} + \lfloor K_p \rfloor \{x_p\} = \{R_p\} - \{\ddot{z}_g\} \tag{2}$$

in which $[M_P]$, $\lfloor C_p \rfloor$ and $\lfloor K_p \rfloor$ denote the mass matrix, the damping matrix and the stiffness matrix of the soil pile foundation system, respectively. $\{R_p\}$ denotes the reaction force due to the interaction force to the superstructure. $\{\ddot{z}_g\}$ denotes a seismic input motion. Taking into accounts for the equiliburium condition and compatibility between the superstructure and soil pile foundation, the governing equation of motion of the total system can be expressed as follows:

$$\begin{bmatrix} [M_{aa}] & [\tilde{M}_{ab}] \\ [\tilde{M}_{ba}] & [\tilde{M}_{bb}] \end{bmatrix} \begin{Bmatrix} \{\ddot{u}_c\} \\ \{\ddot{x}_p\} \end{Bmatrix} + \begin{bmatrix} [C_{aa}] & [\tilde{C}_{ab}] \\ [\tilde{C}_{ba}] & [\tilde{C}_{bb}] \end{bmatrix} \begin{Bmatrix} \{\dot{u}_c\} \\ \{\dot{x}_p\} \end{Bmatrix} + \begin{bmatrix} [K_{aa}] & [0] \\ [0] & [\tilde{K}_{bb}] \end{bmatrix} \begin{Bmatrix} \{u_c\} \\ \{x_p\} \end{Bmatrix} = -\begin{bmatrix} [\tilde{M}_{ab}] \\ [M_p] \end{bmatrix} \{\ddot{z}_g\} \tag{3}$$

in which the matrices $[\tilde{M}_{ab}]$, $[\tilde{M}_{bb}]$ denote the matrices related in terms of interaction effects.

There are several models on evaluation of the nonlinear characteristics of soil subjected to seismic motion. In the present study, the nonlinear soil is represented with the equivalent linerarization (EL) model and Hardin-Drnevich (HD)model. The relation between shear stress and shear strain is evaluated with the EL model and HD model, respectively. For the HD model, the skeleton curve of the stress-strain is given by

$$\tau = \frac{G_0 \gamma}{1 + \gamma / \gamma_r} \tag{4}$$

and the equivalent damping ratio due to soil is expressed by

$$h = \frac{4}{\pi}[1 + r_s]\left[1 - r_s \ln(1 + \frac{1}{r_s})\right] - \frac{2}{\pi}$$

(5)

in which $r_s = \gamma / \gamma_r$ and τ denotes the shear stress, γ the shear strain, G_0 the elastic shear modulus, and γ_r the reference strain (10^{-3}), respectively.

If the dynamic response is dealt within linear responses, the governing equation of the total system can be effectively solved with the modal analysis and the spectrum method. On the other hand, if the response of structure comes to nonlinear situations, it is necessary to determine the dynamic response with the incremental method which can be expressed in terms of the response quantities of the previous step. The incremental method can be thus carried out using step by step integration in the time domain. Applying the incremental method for Eq.(3), it can be expressed with the following equation to each time increment.

$$[\tilde{M}]\{\Delta\ddot{x}\} + [\tilde{C}]\{\Delta\dot{x}\} + [K(t)]\{\Delta x\} = \{\Delta\tilde{F}\} \tag{6}$$

in which $[\tilde{M}]$ and $[\tilde{C}]$ denote the mass matrix and damping matrix including effects on the soil structure interaction. The stiffness matrix $[K(t)]$ also depends upon the structural characteristics and response quantities. Using these results, the equation (6) can be expressed as follows:

$$[\tilde{K}(t)]\{\Delta\ddot{x}\} = \{\Delta\tilde{F}\} \tag{7}$$

in which

$$[\tilde{K}(t)] = [K(t)] + (4/h^2)[\tilde{M}] + (2/h)[\tilde{C}]$$
$$\{\Delta\tilde{F}\} = \{\Delta F\} + [\tilde{M}]\{(4)\dot{x} + 2\ddot{x}\} + [\tilde{C}]\{2\dot{x}\}$$

Solving the equation (7) at each time increment, the response quantities can be determined by these results. Assuming the relation between the bending moment and the curvature depends upon nonlinear characteristics such as a bilinear model, the response can be determined by iteration until performing the convergence of the relations. In the present study, the nonlinear characteristic of the structure is given by the $M-\phi$ relation. The iterative procedure can be carried out using the Newton Raphson method.

RESULTS AND DISCUSSIONS

Input seismic motions and strength demand spectra

From the design points of views on the structure subjected to severe seismic motion, it is available to evaluate the maximum response quantities of the structure by means of a convenient method. The maximum response of the nonlinear system can be easily determined with the strength demand spectra, which are determined by the seismic response of the SDOF system with the nonlinear spring, such as the bilinear model. The nonlinear characteristic of the steel structure can be represented with the bilinear model.

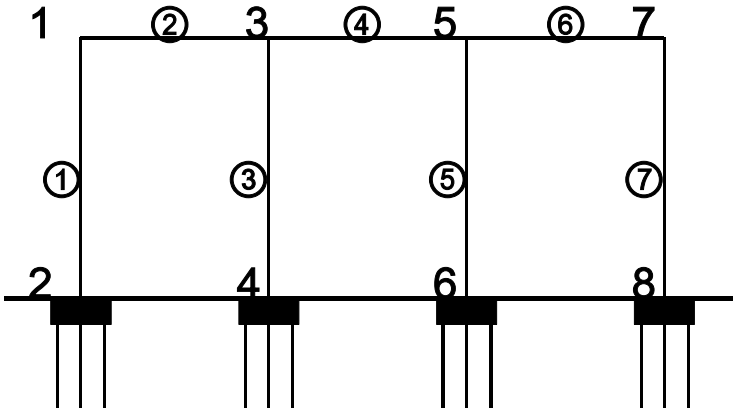

Fig.1 An idealized model of a bridge pier foundation system

Fig.1 shows an idealized model of a bridge pier foundation system. Each pier height has various one such as taking the natural period of the structure from 0.3 sec and 1.0 sec. The critical damping ratio of the first mode is 2%. The soil pile foundation system is represented with a sway rocking (SR) model. The spring constants with respect to soft and hard soil conditions are given by Table 1. The seismic response analyses are carried out using three components of seismic motions, which are Kobe ns (1995,M7.2) at relatively hard soil condition, Taka ns (JR Takatori (1995,M7.2)) at soft soil condition and Akune (Kagoshima (1997), M6.5) at relatively soft soil condition.

Fig.2 shows the response spectra of accelerations corresponding to these seismic motions. The maximum accelerations of these seismic motions are normalized by 500 gal. Namely, the response due to Kobe ns component dominates in the natural periods from about 0.3 sec to about 0.7 sec. The response due to Taka ns component dominates in the natural periods from 0.4 sec to about 1.5 sec. The response due to Akune ns also yields several peaks in the natural periods from about 0.25 sec to about 1.5 sec. It is understood that the response spectra mainly depends upon the dynamic characteristics of dominant soil situation.

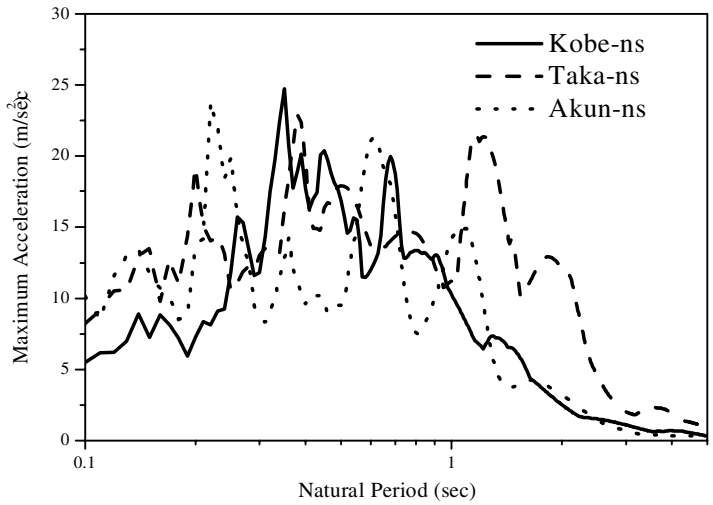

Fig.2 Response spectra of accelerations

Table 1 Initial stiffness of piles

(x10^5)

Soil conditions	sway(kN/m)	rocking(kNm/rad)
soft	6.91	42.8
hard	35.6	65.9

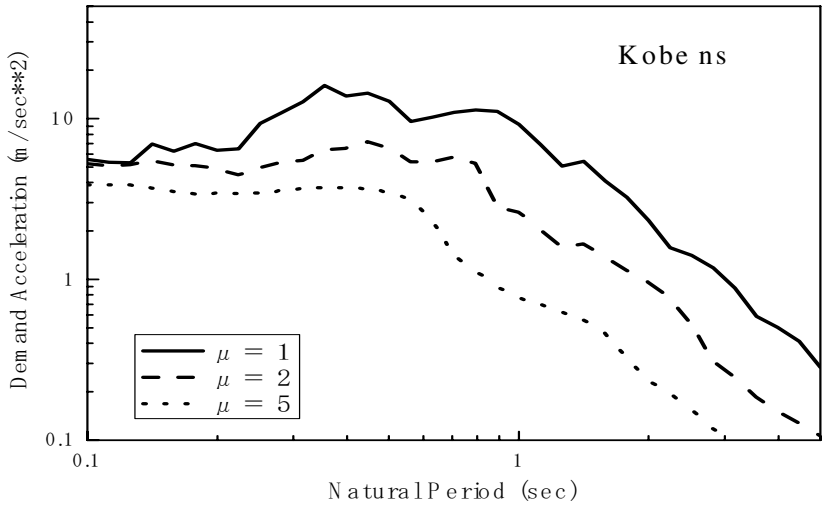

Fig.3 Strength demand spectra

If the structure can be represented with the SDOF system, the strength demand spectra play important roles on the nonlinear response evaluations. Fig.3 shows the strength demand spectra for the seismic motions of Kobe ns. It is understood that as increasing the ductility ratio, the required yield strength of the structure can be particularly reduced in the most important frequency region. The strength demand spectra

are very helpful to evaluate the nonlinear response, but it is necessary to examine how to apply to the MDOF system. If the structure response is mainly depends upon the dominant response properties such as the first vibration mode, the seismic response analysis can be implemented with the strength demand spectra for the framed structure with the soil pile foundation. In order to examine nonlinear response characteristics of the framed structure with the soil foundation system, nonlinear seismic response analyses to various soil conditions are carried out using the strength demand spectra and the incremental method as expressed in Eq.(7).

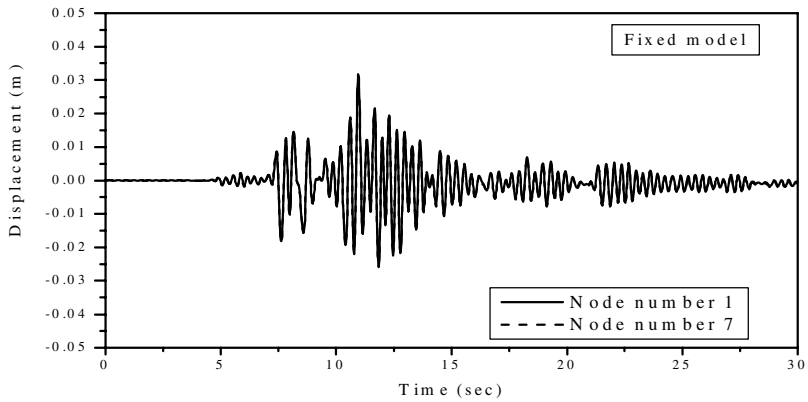

Fig.4 Time history of displacement responses

Dynamic response properties

Fig.4 shows the time histories of displacement responses of the structure at the nodal point 1 and 7 to the system with the fixed conditions on the soil foundation. The natural period of the structure is 0.32 sec. It is noted that the horizontal displacement is governed at the girder response and is mainly depended upon the first vibration mode.

Fig.5 also shows the time histories of displacement responses at the nodal point 1 and 7 to the system with the soil pile foundation. In this case, the superstructure of the system is taken into accounts for being a nonlinear situation and the soil condition being a linear one. It is noted that as a whole the displacement response at the two points become to be similar ones for the case with the soil structure interaction and the response corresponding to the first vibration mode plays important roles on the response evaluation.

Fig.6 also shows the hysteresis between the bending moment and curvature at the element 1 and element 7 to the fixed foundation condition, respectively. The nonlinear characteristic of $M-\phi$ relation is a bilinear and the hysteresis of the piers has similar ones. It is understood that the response corresponding to the first vibration mode comes out the dominating effect on the seismic responses.

Fig.7 similarly shows the hysteresis on the bending moment and curvature to the soil structure interaction system. It is noted that the dissipation energy due to the soil foundation system plays some contributions on reducing the responses of the superstructure. For the total structure system subjected to seismic forces, it is understood that complicate properties on the responses are brought out by nonlinear characteristics. If the response due to seismic motions mainly depends upon the dominate vibration mode, it is supposed that the strength demand spectra give efficient methods on the nonlinear response evaluations. Since the intensity and characteristics of input seismic motions and the dynamic properties of the structure yield important influences on the responses, it is certainly important to make the nonlinear response clear for the MDOF system.

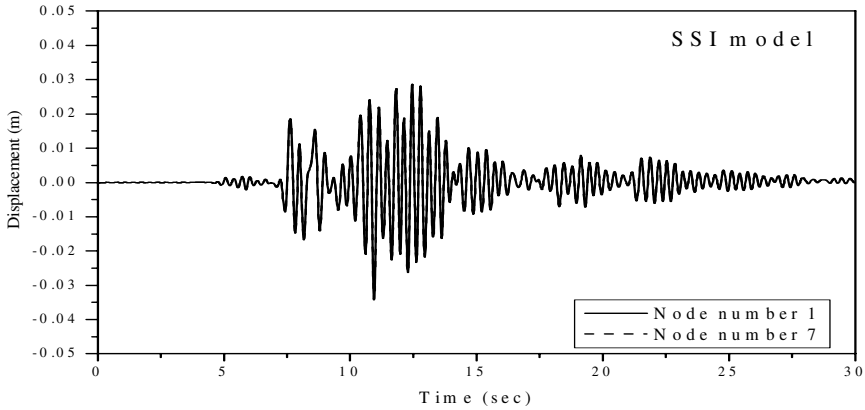

Fig.5 Time history of displacement response

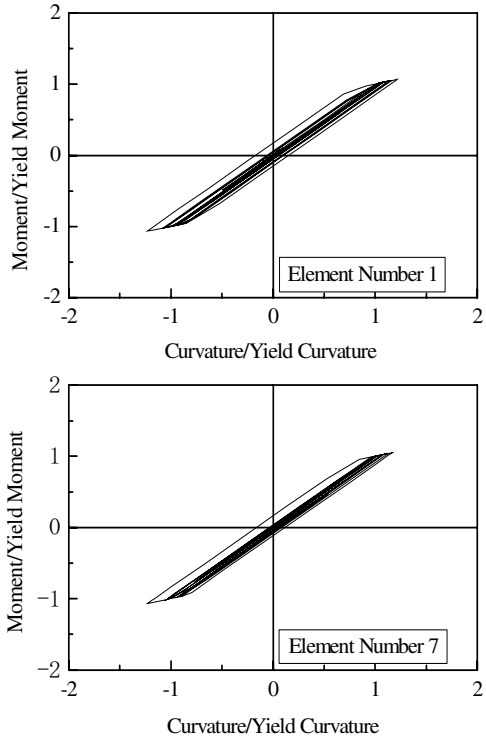

Fig.6 Hystersis on bending moment and curvature(fixed model)

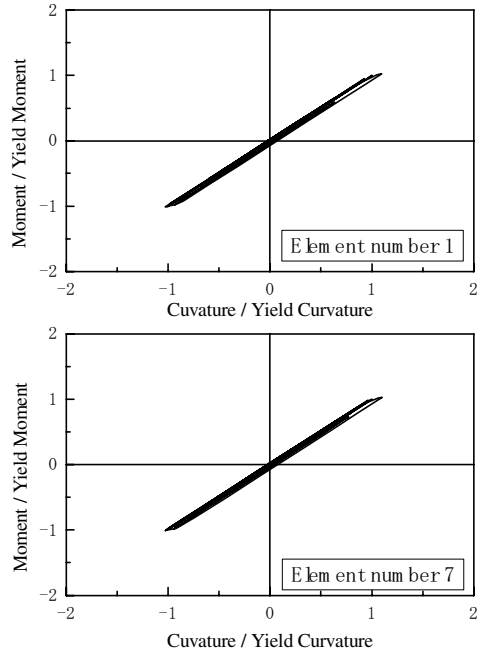

Fig.7 Hysteresis on bending and curvature (SSI model)

Comparisons of responses of MDOF system with SDOF system

Fig.8 shows allowable accelerations of the structure for required ductility ratios with respect to the fixed foundation. Comparison is made for the response between the SDOF system and the MDOF system. The result on the SDOF system is obtained by means of the strength demand spectra. There are some

differences on the responses of the ductility ratio, 5, to these systems. For the MDOF system, the dynamic response properties due to nonlinearity turns out different situations with respect to responses and the section force such as bending moment is redistributed by means of the nonlinearity. In the present case, the allowable acceleration to the MDOF system has a tendency to become larger than the SDOF system. It is understood that the allowable acceleration of the MDOF system cannot be always evaluated with only the strength demand spectrum.

Fig.9 shows ratios of the allowable acceleration of the structure on the ductility ratio, 5, to the ductility ratio, 1, for the Kobe ns component. It is shown together with the responses with respect to the soil structure system, such as SSI model, E-L model, and H-D model, respectively. The SSI model denotes the linear soil structure interaction model, the E-L model the equivalent nonlinear soil structure interaction model, and the H-D model the Hardin-Dernevich nonlinear soil structure interaction model, respectively. In spite of the soil structure interaction, the response ratio turns out comparable results for the MDOF system model. However, it is noted that there is considerable differences between the SDOF system and the MDOF system to the present situations.

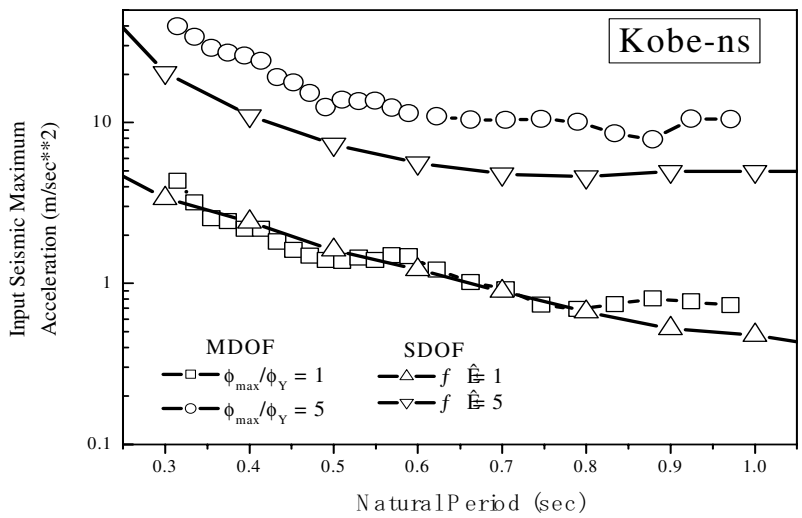

Fig.8 Allowable accelerations for demand ductility ratio (fixed model)

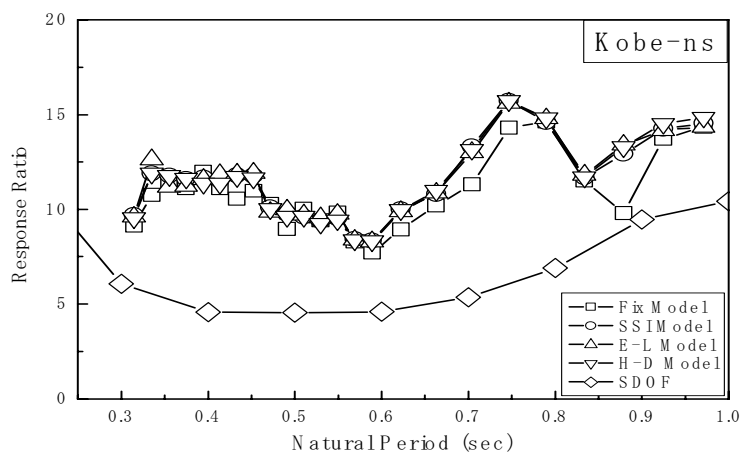

Fig.9 Comparison of allowable accelerations for nonlinear model

Fig. 10 similarly shows comparisons on the allowable acceleration of the structure with respect to the response of the ductility ratio, 5, to the ductility ratio,1. The input seismic motion is the Taka ns component. There are relatively small differences among the MDOF system with respect to the soil structure interaction. However, the allowable accelerations of the MDOF system as a whole come out larger ones than the SDOF system. It is understood that the results on the SDOF system by means of the strength demand spectra give considerably safer evaluations than the MDOF system. The SDOF system as a whole gives smaller allowable acceleration than the MDOF system.

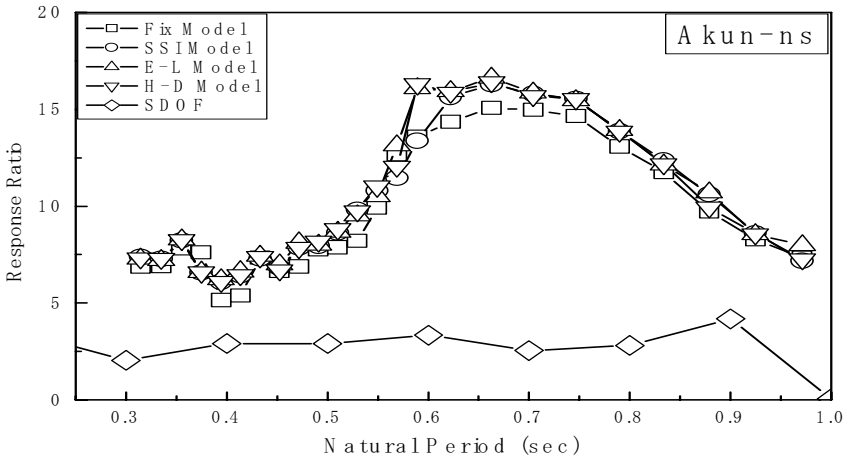

Fig.10 Comparisons of allowable acceleration for nonlinear model

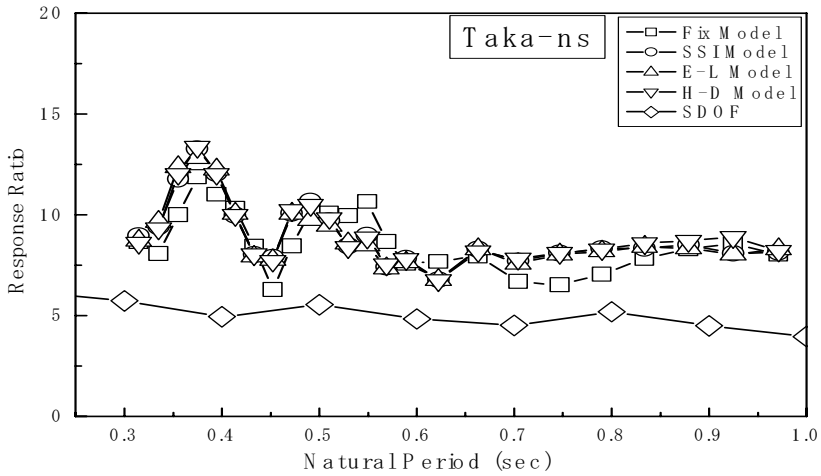

Fig.11 comparisons of allowable acceleration for nonlinear models

Fig.11 similarly shows comparisons on the allowable acceleration ratio of the ductility ratio, 5, to the ductility ratio, 1. The input seismic motion is the Akune ns component. As previously mentioned, there are small differences among the MDOF system with the soil structure interaction. In the present case, the SDOF system turns out considerably smaller results than the MDOF system. It is noted that the allowable

Table 2 Damage level to seismic forces

Damge	drift/ pier height
severe	1/ 100
moderate	1/ 150
small	1/ 300
negligible	1/ 1000

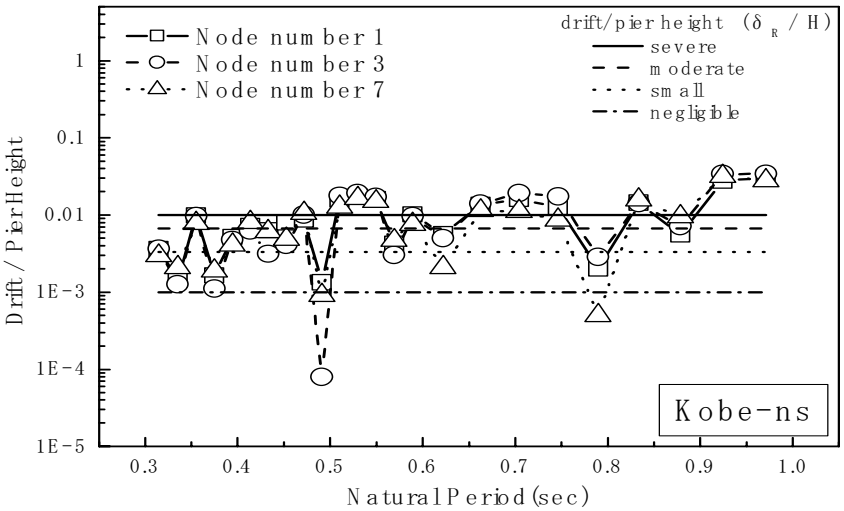

Fig.12 Drift displacement spectra to the total system

input acceleration has properties depending upon not only the nonlinear model on the MDOF system but also characteristics of input seismic motions.

Damage assessment

From the design points of views of the structure subjected to severe seismic motions, it is very important to make the damage assessment clear by means of the available index. While several damage indices are proposed by many researchers, it has been also continued to develop more available index on the seismic damage assessment. After the structure subjected to severe seismic forces, it is supposed to have caused by some damages to the structure. These damages may be appeared by some deformations of the structure such as the drift, which is useful one of the most simplified damage assessments. Table 2 describes an example of the damage assessment on the bridge structure. The damage level is assessed by the ratio of the horizontal drift to the height of the pier. The ratio over 0.01 corresponds to have caused to severe damage.

Fig.12 shows the drift displacement spectra for the response of structure to the Kobe ns component. In order to examine the damage evaluation by drift displacement for various type of the structure, it is dealt with the superstructure which has the natural periods from 0.3sec to 1.0 sec. The results on the pier 1, pier 3 and the pier 7 yield different response ratios to each natural period of the structure. While some piers have caused severe damages, the other pier turns out very small damages by depending upon the natural

period of the structure. It is understood that since the damage ratio due to drift displacement comes to various ones with respect to response characteristics of the structure, it is necessary to assess the damage by means of the response of the total system.

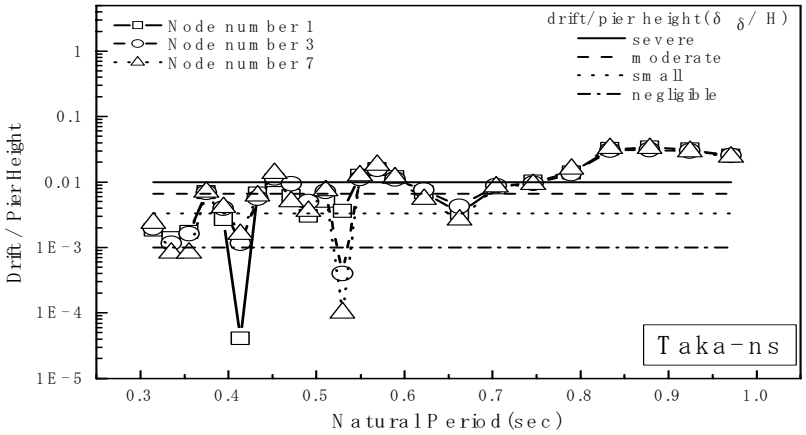

Fig.13 Drift displacement spectra to the total system

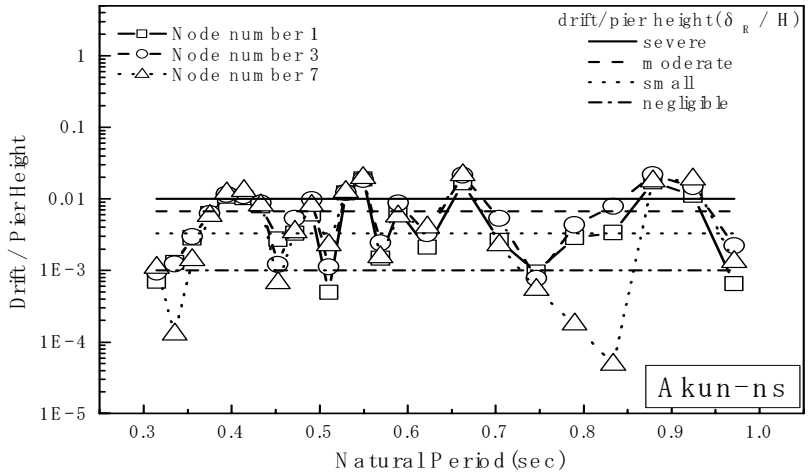

Fig.14 Drift displacement spectra to the total system

Fig.13 similarly shows the drift displacement spectra of the structure subjected to the Taka ns component. The results are denoted at the same points as the previous case. There are some variations on the drift displacement to each natural period. It is especially noted that the drift displacement with respect to the natural period under about 0.5 sec turns out different one at each pier. For the soil structure interaction system, it is important to carry out the damage assessment by means of the MDOF system.

Moreover, the drift spectra of displacement due to the Akune ns component are similarly shown in Fig.14. The damage assessment due to the drift displacement turns out different one depending upon the natural periods. Since the damage assessment comes to various results at each pier, it is understood that it should be carried out the damage evaluation due to the total structure system. Since the Akune ns component includes more broad range of dominating frequency than the other seismic motion, the damage

assessment seems to yield some different results from the other cases of the seismic motion. It is suggested that in order to carry out an available damage assessment of the structure, it is important to assess the damage situation by means of not only the SDOF system but also the MDOF system.

CONCLUSIONS

The damage assessment due to the drift of displacement is carried out to a bridge structure in the present study. The main results are summarized as follows:

(1) Comparing the MDOF system with the SDOF system for the nonlinear response situation, the MDOF system can take larger allowable input intensity of seismic motion than the SDOF system for the demand ductility ratio. It is suggested that the strength demand spectra on the SDOF system gives generally weaker strength than the MDOF system.
(2) The nonlinear response characteristics of the MDOF structure with the soil foundation system yield more complicate ones than the SDOF system. In order to carry out the damage assessment of the structure, it is definitely essential to make the nonlinear response characteristics clear by examinations of the MDOF system.
(3) The drift displacement spectra of the MDOF system are mainly dependent upon the natural period of the structure system and input seismic motion characteristics. Since the damage assessment of the structure by means of the drift of the pier comes out various values at each pier, it is suggested that the more available evaluation can be obtained through examinations with respect to the MDOF system with soil foundation system.

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