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STATISTICAL SEISMIC PERFORMANCE OF STEEL BRIDGE PIERS CONSIDERING VARIATION OF STRUCTURAL PROPERTIES

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SUMMARY

Recently, in the world, the performance-based design has been accepting as seismic design method of civil engineering structures. Moreover, International Organization for Standardization (ISO) has been constructing the general principals for design method of structures. The concept of these design methods is based on reliability theory. Besides the importance of seismic performance check by dynamic analyses has been increasing after the great disaster due to the Hyogo-ken Nanbu Earthquake in 1995 in Japan. These situations require estimating exactly the nonlinear seismic performance of structures. Therefore, it is very important to investigate the influence of uncertainty of structural properties on non-linear dynamic behaviors of structures. In this study, dynamic analyses of steel bridge piers subjected to simulated waves were performed considering the variation of structural properties. In order to clarify the influence of uncertainty of structural properties on non-linear dynamic behaviors, width-thickness parameter, slenderness parameter, and damping ratio are considered as the structural properties. Moreover, the difference of the results between in case subjected the simulated waves of plate boundary type earthquake (so-called Type-I by Japan Seismic Design Specifications of Highway Bridges) and in case subjected to the waves of inland earthquake (so-called Type-II) is investigated. As a result, it is found that, irrespective of the type of input earthquake waves (such as Type-I and Type-II), the coefficient of variation is from about 10% to 14% for the maximum response displacement and is from about 4% to 10% for the maximum response horizontal force. Consequently, we proposed the estimation equation for the maximum response displacement considering the variation dependent on uncertainty of structural properties.

INTRODUCTION

Recently, performance-based design has been accepted as a seismic design method of civil engineering structures in JSSC [1]. The International Organization for Standardization (ISO [2]) has been constructing general principals for the design method of structures. The concept of these design methods is based on reliability theory.

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Moreover, the importance of seismic performance check by dynamic analyses has been increasing due to the catastrophic Hyogoken-Nanbu Earthquake in 1995 in Japan. These situations require estimating exactly the nonlinear seismic performance of structures (for example AIJ [3] and JSCE [4]).
Before now, the influences of the variation of material properties on the ultimate strength and the buckling strength of structures have been investigated widely. As the design method considering these influences, plural formats for limit state design code have been proposed in AIJ [5, 6] and JSCE [7].
However, in these formats, only the static ultimate strength is considered as the limit states of structures. Moreover, the load effects are basically obtained from the static analysis. As has already mentioned, it has been important to check dynamic performances of structures exactly in the seismic design. Therefore, it is very important to investigate the influence of uncertainty of structural properties on non-linear dynamic behaviors of structures.
In this study, dynamic analyses of steel bridge piers subjected to simulated waves were performed considering the variation of structural properties. In order to clarify the influence of uncertainty of structural properties on non-linear dynamic behaviors, generalized width-thickness parameter, generalized slenderness parameter, and damping ratio are considered as the structural properties.
Moreover, the difference of the results between one case subjected to the simulated waves of plate boundary type earthquakes (so-called Type-I by JRA [8]) and another case subjected to the waves of inland earthquakes (so-called Type-II) is investigated.

ANALYSIS METHOD

Analysis Targets and Models

Analysis targets are nine steel bridge piers as shown in Table 1. These bridge piers are designed on the type II ground by Design Specifications of Japan Road Association 2000. The steel piers are stiffened box sections. The width-thickness parameter and the slenderness parameter of steel piers are set at 0.3 to 0.6 and 0.25 to 0.65, respectively. Natural periods of steel bridge piers are from 0.38 to 1.41 second. The width-thickness parameter and the slenderness parameter are defined as follows:

Table 1 Target bridge piers

Model Name	Width-Thickness Parameter	Slenderness Parameter	Natural Period
S3025	0.30	0.25	0.38
S3045	0.30	0.45	0.69
S3065	0.30	0.65	1.02
S4525	0.45	0.25	0.47
S4545	0.45	0.45	0.86
S4565	0.45	0.65	1.26
S6025	0.60	0.25	0.53
S6045	0.60	0.45	0.96
S6065	0.60	0.65	1.41

$$R_f = \frac{b_f}{t}\sqrt{\frac{\sigma_y}{E}\cdot\frac{12(1-\mu^2)}{\pi^2 k}} \quad (1)$$

$$\bar{\lambda} = \frac{2h}{r}\cdot\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}} \quad (2)$$

where b_f=flange plate width, t=flange plate thickness, σ_y=yield stress, E=Young's modulus, μ=Poisson's ratio, $k=4n^2$: buckling coefficient, h=length of member, n=number of sub panel girded by stiffening member and r=radius of gyration.

The single degree-of-freedom model is adopted as shown in Fig. 1 because the target structures are single column type bridge piers which are popular in Japan. The base of the models is fixed in the analysis.

As the hysteresis characteristics of steel piers, a 2-parameter model proposed by Suzuki [9] is used. This model can represent the degrading of stiffness and strength with local buckling of plates. The skeleton curve is assumed as a tri-linear model as shown in Fig. 2. In Fig. 2, the example of a hysteresis loop of S6025 is presented. The vertical axis shows the generalized horizontal force by the yield horizontal force H_y and the horizontal axis shows the generalized horizontal displacement by the yield horizontal displacement δ_y. H_m and δ_m in Fig. 2 indicate maximum horizontal force and maximum horizontal displacement, respectively.

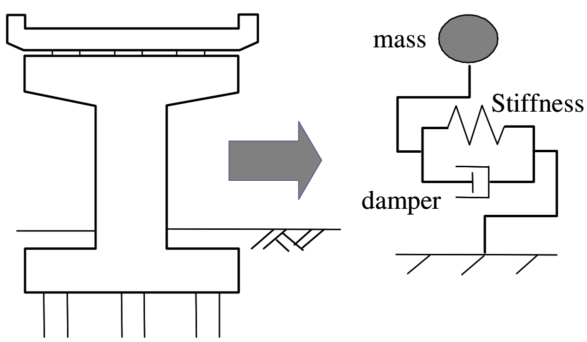


Figure 1 Analysis model

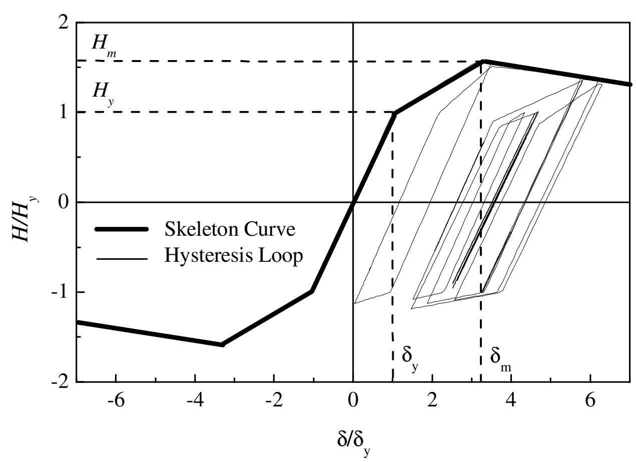


Figure 2 Hysteresis characteristics

Input Earthquake Motions

In this study, artificial seismic waves are used as input earthquake motions. Artificial seismic waves are generated as follows:

1. The spectrum of the Level 2 (for estimating the ultimate limit state) Type I (plate boundary type earthquake of large scale) and Type II (inland earthquake, such as Hyogoken-Nanbu Earthquake) input earthquake motions shown in Fig.3 (JRA [8]) are set as the target spectrum.
2. Simulated waves are calculated using the synthetic method by superposing sinusoidal waves.
3. The duration characteristics are considered by the Jennings type envelope curve.

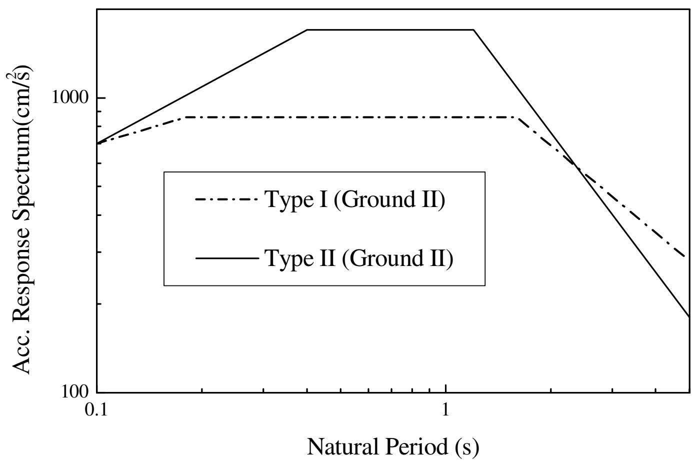
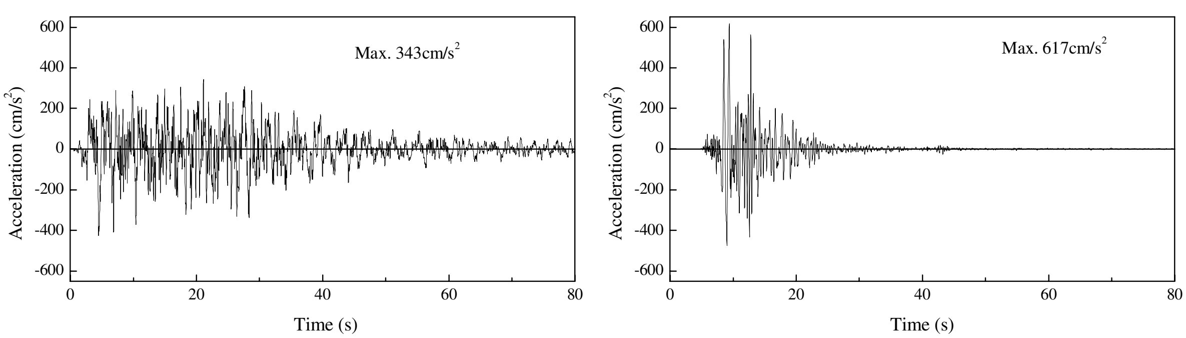


Figure 3 Target spectra

In this case, the phase characteristics are given as random values. In Fig. 3, the dashed-dotted line indicates the design spectrum for Type I input motions and the solid line indicates one for Type II. The acceleration time histories of generated artificial waves are shown in Fig. 4. In Fig. 4, (a) shows the time history of Type I and (b) shows that of Type II. These figures show that the Type I wave has a long duration time of the principal motion and no clear peak and that the Type II wave has a short duration time of the principal motion and a clear peak.



(a) Type I wave (b) Type II wave

Figure 4 Acceleration time history

VARIATION FACTORS

Material Properties for Variation Factors

The following parameters are considered as the factors of steel bridge piers. Those are yield stress σ_y, ultimate stress σ_u, Young's modulus E, Poisson's ratio ν, thickness of plate t, width of plate b, length of member h, radius of gyration r and vertical force P.

In reference to JSCE [7], Itoh [10] and AIJ [5], the characteristics of variation factors shown in table 2 are assumed. In Table 2, F_y, F_u and P_y are nominal yield stress, nominal ultimate stress and yield vertical force, respectively. E_n is $2.06*10^5$ N/mm^2, ν_n is 0.3 and, t_n, b_n, h_n, and r_n are each mean value.

It is likely that there are some factors which have a correlation among themselves. However, it is difficult to quantitatively estimate the correlation between them. Hence, we assume that they are independent of one onother.

1,000 random values for each factor are generated. The variation characteristics of the generated random values are shown in Table 3. Comparing Table 2 with Table 3, the errors are not more than about 0.4% for every factor.

Table 2 Assumed probabilistic characteristics of variation factors

	σ_y/F_y	σ_u/F_u	E/E_n	v/v_n	t/t_n	b/b_n	h/h_n	r/r_n	P/P_y
Mean	1.390	1.092	0.999	0.937	1.000	1.000	1.000	1.000	1.050
Standard Dev.	0.161	0.068	0.045	0.085	0.010	0.010	0.010	0.010	0.010
C.O.V.	0.116	0.062	0.045	0.091	0.010	0.010	0.010	0.010	0.010
Distribution	Log. Normal	Normal	Log. Normal	Log. Normal	Normal	Normal	Normal	Normal	Normal

Table 3 Probabilistic characteristics of random value for variation factors

	σ_y/F_y	σ_u/F_u	E/E_n	v/v_n	t/t_n	b/b_n	h/h_n	r/r_n	P/P_y
Mean	1.404	1.091	0.999	0.945	1.000	1.000	1.000	1.000	1.048
Standard Dev.	0.160	0.065	0.046	0.081	0.010	0.010	0.010	0.010	0.010
C.O.V.	0.114	0.062	0.046	0.090	0.010	0.010	0.010	0.010	0.010
Distribution	Log. Normal	Normal	Log. Normal	Log. Normal	Normal	Normal	Normal	Normal	Normal

Probabilistic characteristics of structural properties

M_y=yield moment, h=length of member, P=vertical force, P_y=yield vertical force and EI=bending stiffness. Using the estimation equations by Suzuki [9], maximum horizontal force H_{max} and maximum horizontal displacement δ_{max} are calculated by the following equations:

$$\frac{H_{\max}}{H_y} = 0.101\left(R_f \cdot \bar{\lambda}\right)^{-1.0} + 0.880 \tag{3}$$

$$\frac{\delta_{\max}}{\delta_y} = 0.00759\left(R_f \cdot \sqrt{\bar{\lambda}}\right)^{-3.5} + 2.59 \tag{4}$$

The simulated results are shown in Table 4. In Table 4, the mean value of every structural property is normalized to 1.0. It is found that the distribution of every structural property fits a log-normal distribution and the coefficient of variation is from 0.06 to 0.12.

However, yield horizontal force, yield horizontal displacement, maximum horizontal force and maximum horizontal displacement are strongly correlated with width-thickness parameter and slenderness parameter as evidenced by Eqs. (3) and (4). Therefore, in the variation analysis, width-thickness parameter, slenderness parameter and damping ratio are considered as variable factors.

Table 4 Probabilistic characteristics of simulated value for structural properties

	R_f	$\bar{\lambda}$	H_y	δ_y	H_{max}	δ_{max}
Mean	1.000	1.000	1.000	1.000	1.000	1.000
Standard Deviation	0.068	0.063	0.118	0.124	0.181	0.635
C.O.V.	0.068	0.063	0.118	0.124	0.181	0.635
Distribution	Log. Normal	Log. Normal	Log. Normal	Log. Normal	Log. Normal	Log. Normal

Variation Analysis Model

In the variation analysis, a 2-point estimation method (Rosenblueth [11]) is adopted because the numbers of analysis cases are very huge using the simple Monte Carlo Method. A second order moment of probabilistic values is required in the 2-point estimation method. The concept of the 2-point estimation method is shown in Fig. 5.

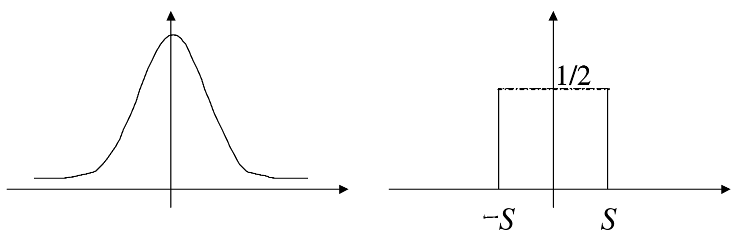


Figure 5 Concept of 2-point estimation method

In this study, the mean value M and the coefficient of variation S are calculated from the analysis results of 8 cases shown in Table 5.

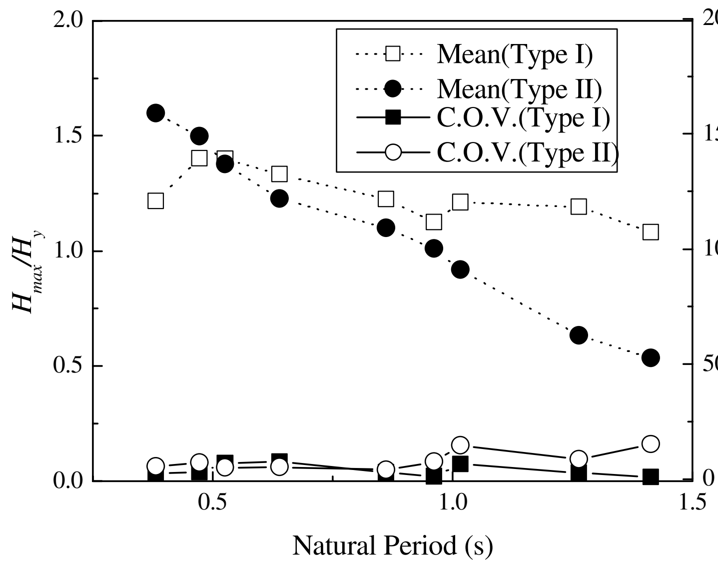
Table 5 Analysis cases

Case	1	2	3	4	5	6	7	8
Width-Thickness Parameter	(1+S)M	(1+S)M	(1+S)M	(1+S)M	(1-S)M	(1-S)M	(1-S)M	(1-S)M
Slenderness Parameter	(1+S)M	(1+S)M	(1-S)M	(1-S)M	(1+S)M	(1+S)M	(1-S)M	(1-S)M
Damping Ratio	(1+S)M	(1-S)M	(1+S)M	(1-S)M	(1+S)M	(1-S)M	(1+S)M	(1-S)M

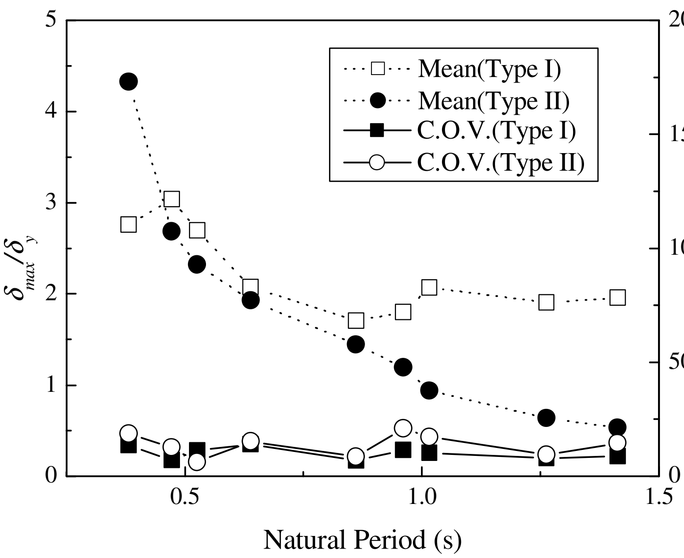
RESULTS AND DISCUSSIONS

Results of the Variation Analyses

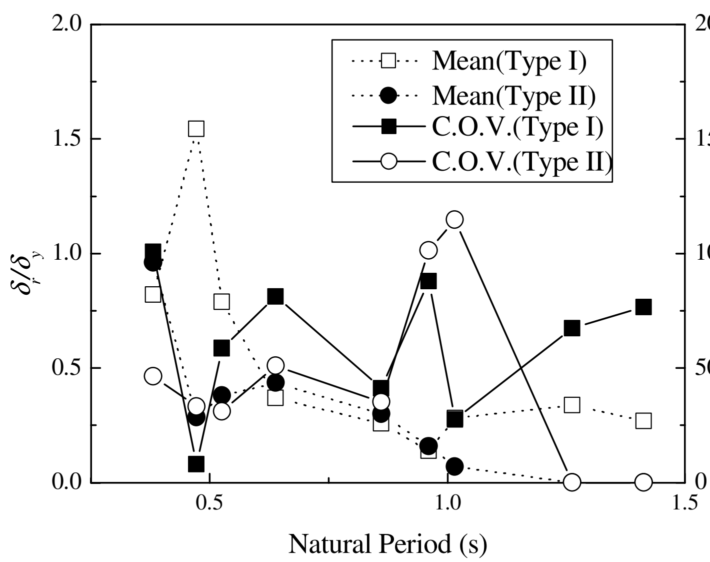
The results of the variation analyses for maximum response horizontal force, for maximum response horizontal displacement and for residual displacement are shown in Figs. 6(a), (b) and (c), respectively. The dotted lines with open squares and with solid circles indicate the mean values subjected to Type I wave and subjected to Type II wave, respectively. The solid lines with close squares and with open circles indicate the coefficients of variations subjected to Type I wave and subjected to Type II wave, respectively.



(a) Horizontal force



(b) Displacement



(c) Residual displacement

Figure 6 Variation analysis results

In Figs. 6(a) and (b), the results for maximum horizontal response force are similar to ones for maximum horizontal response displacement. Moreover, the mean values are much different according to the natural periods of structures.
However, the coefficients of variation do not vary according to the natural periods of structures. The coefficients of variation for maximum horizontal force and for maximum horizontal displacement are from 4 to 8% and from 10 to 14%, respectively. It follows from what has been said that the variation characteristics for maximum response are kept at a constant state irrespective of the natural periods of structures.
On the other hand, from Fig. 6(c), it is found that the mean value and the coefficient of variation for residual displacement vary greatly according to the natural periods of structures.

Application to Seismic Design

We proposed an estimation method for the maximum response horizontal displacement using the Natural-Period-Dependent Spectrum Intensity (Kitahara [12, 13 and 14]). However, the equation leaves the influence of the variation of structural properties out of consideration. Therefore, the following new equation is proposed considering the variance of maximum response displacement due to variations of structural properties.

$$\delta_{design} = \gamma_{str.} \frac{T_{eq}}{2\pi} \cdot SI_{n.p.} \tag{5}$$

where, δ_{design}=design value of the maximum response displacement, $\gamma_{str.}$=structural property coefficient constant, T_{eq}=equivalent natural period, and $SI_{n.p.}$=natural-period-dependent spectrum intensity.
The structural property coefficient $\gamma_{str.}$ is the constant representing the influence of the variation of structural properties. In section 4.1, it was shown that the coefficients of variation are from 10 to 14% irrespective of the natural periods of steel bridge piers. Therefore, this study suggests that the structural property coefficient constant should be set at about 1.2 considering the variation of the maximum response displacement.

CONCLUSIONS

The following conclusions may be drawn from the present study:

(1) The maximum response horizontal force varies from 4 to 8% irrespective of the natural periods of steel bridge piers considering the realistic variation of structural properties such as with-thickness parameter, slenderness parameter and damping ratio.
(2) The maximum response horizontal displacement varies from 10 to 14% irrespective of the natural periods.
(3) The design equation for the maximum response displacement considering the variance due to the variation of structural properties as a partial safety coefficient is proposed.

Whereas, the investigation considering the uncertainty of input earthquake motions due to its non-stationary characteristics is future problem.

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