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CORRELATION BETWEEN NONLINEAR RESPONSE OF BRIDGE PIERS AND NATURAL-PERIOD-DEPENDENT SPECTRUM INTENSITY

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SUMMARY

In general, the results of nonlinear dynamic analyses usually depend on the periodic characteristics of input earthquake waves. Therefore, it is very important to investigate the relevance of the nonlinear dynamic response of structures and the index of seismic motions. In order to study which index of seismic motions has a strong correlation to the nonlinear dynamic response of bridge piers, the elasto-plastic dynamic analyses using the SDOF model are carried out with observed waves and simulated waves. The maximum response displacement and the residual displacement are considered as the dynamic responses of bridge piers, while the peak ground acceleration (PGA), the peak ground velocity (PGV), the peak ground displacement (PGD) and the spectrum intensity (SI) are considered as the basic indices of seismic motions. However, the correlation between the dynamic responses and the basic indices of seismic motions varies according to the natural period T of bridge piers. Thus, modified spectrum intensity, called natural-period-dependent SI, is proposed in this paper as the effective index of seismic motions. It can be calculated by the integration of the velocity response spectrum in the range corresponding to the natural period of the target piers. In this study, it is found that the optimum integration ranges for natural-period-dependent SI are from $0.9T$ to $1.2T$ for steel piers and from $1.0T$ to $2.8T$ for RC piers, respectively. The difference of integration ranges between steel and RC piers comes from the difference of prolongation of natural period in nonlinear region of structures. Consequently, it is clarified that the correlation between dynamic maximum response and natural-period-dependent SI is very strong over a wide range of natural period.

INTRODUCTION

The Hyogoken-Nanbu Earthquake in 1995 caused significant damages of civil engineering structures. Since the earthquake, the Japan Society of Civil Engineers (JSCE) has published the proposal on earthquake resistance for civil engineering structures [JSCE, 1996]. The Architectural Institute of Japan (AIJ) has also announced the proposal on improvement of disaster prevention for architecture and city [AIJ, 1998]. In those proposals, it has been recognized that the importance of a dynamic analysis has increased in the seismic design of structures.

This research aims to study which index of seismic motions has a strong correlation to the nonlinear dynamic response of bridge piers. Because the characteristics of seismic motions are strongly dependent on the results of nonlinear dynamic analyses, a result of dynamic analysis is usually different from another result using different wave with the same peak ground acceleration. Therefore, it is very important that the relevance of the nonlinear dynamic response and the index of seismic motions is investigated to reduce the variance of the numerical analysis results and to select adequate input waves considering the periodic characteristics.

As the basic indices of seismic motions, the peak ground acceleration (PGA), the peak ground velocity (PGV), the peak ground displacement (PGD) and the spectrum intensity (SI) have been considered. A large number of studies have been made on the correlation between these basic indices of seismic motions and dynamic responses of structures [Nagahashi and Kobayashi, 1971, Kitahara and Itoh, 1998]. These studies presented that

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the indices, which have strong correlation to dynamic response, are the PGA in the short period range, the PGV in the middle period range and the PGD in the long period range. However, the question is that the effective index varies according to the natural period of structures. Thus, it is important to investigate a new index which has strong correlation to dynamic response over a wide range of natural period.

In this study, modified spectrum intensity called natural-period-dependent SI is proposed as the effective index of seismic motions. It can be calculated by the integration of the velocity response spectrum in the range corresponding to the natural period of the target structures. Analysis target structures are steel bridge piers and RC bridge piers. In order to study the relevance of dynamic response and the index of seismic motions, the elasto-plastic dynamic analyses using the SDOF model are carried out with observed waves and simulated waves. The maximum response displacement, which is the important parameter in the seismic design, and the residual displacement [Kawashima et al., 1998] which importance is increasing in functional check are considered as the dynamic response of bridge piers.

ANALYSIS METHOD

Input Earthquake Wave

The observed waves and simulated waves in the Design Specifications of Highway Bridges [Japan Road Association, 1996] are used as input earthquake waves. **Table 1** shows the features of input earthquake motions. The original intervals of each seismic wave shown in **Table 1** are 0.01second or 0.02 second. All data are used as 0.005 second interval data by linear interpolation.

Table 1: Earthquake motions

Earthquake	Station	Abbreviation	Type	Direction
Hyogoken-nanbu, Japan	JMA-Kobe	JMA-NS	Near Field	NS
	JMA-Kobe	JMA-EW	Near Field	EW
	Fukiai	FUKI-x	Near Field	(x)
	Fukiai	FUKI-y	Near Field	(y)
	JR Takatori	TAK-NS	Near Field	NS
	JR Takatori	TAK-EW	Near Field	EW
Imperial Valley, USA	El Centro	EL-NS	(Far Field)	NS
	El Centro	EL-EW	(Far Field)	EW
Kern County, USA	Taft	TAFT-NS	Far Field	NS
	Taft	TAFT-EW	Far Field	EW
Tokachioki, Japan	Hachinohe	HACHI-NS	Far Field	NS
	Hachinohe	HACHI-EW	Far Field	EW
(Simulated Wave)	(Type 1)	Typ1	Far Field	-
	(Type 2)	Typ2	Near Field	-

In this study, the PGA, PGV, PGD and SI are first considered as the basic indices of seismic motions. Time histories of velocity and displacement are calculated from acceleration time history using fast Fourier transformation (FFT) integration. SI is calculated by integration of velocity response spectrum.

Proposed Spectrum Intensity

Housner defined that the integration range of velocity response spectrum was from 0.1 to 2.5 second because he assumed that natural periods of the general structures were from 0.1 to 2.5 second [Housner, 1952 and 1965]. Thus, a seismic wave has a unique SI value independent of target structures and SI has generality as index of seismic motions.

The response spectrum at the neighborhood of natural period is meaningful against a certain structure. However, the integration range of SI extends to the region unrelated with the natural period. Therefore, the periodic characteristics against a certain structure are not effectively innovated into SI. That is to say, SI is the effective index of seismic motions in order to evaluate the mean responses of many structures. However, SI is not the effective index of seismic motions to estimate responses of a certain structure.

In this study, natural-period-dependent SI, which is calculated by the integration of the velocity response spectrum in the range corresponding to the natural period of structures, is proposed and presented as Eq. 1 and **Fig. 1**. For instance, in **Fig. 1**, two shaded areas indicate $SI_{n.p}$ corresponding to structures of natural period T_1 and T_2, respectively.

$$SI_{n.p.} = \frac{1}{(\beta-\alpha)T}\int_{\alpha T}^{\beta T} S_v(\tau, h)d\tau \tag{1}$$

where, $SI_{n.p.}$: natural-period-dependent SI, S_v: velocity response spectrum, h: damping ratio,

τ : integration parameter (natural period), T : natural period of the target structure, α,β: constant

It is generally accepted that the natural period of structures is prolonged because the stiffness of structures is degraded with damage. Therefore, it is likely that α =1.0 and β >1.0 in Eq. (1) are the optimal values. However, it is possible that the component in the short period range affects on the dynamic response due to the effect of damping and so on. Thus, the optimal α and β are evaluated from the results of nonlinear dynamic analyses against steel and RC bridge piers. Further, the effectiveness of natural-period-dependent SI is discussed.

Analysis Model

Analysis targets are nine steel bridge piers and five RC bridge piers as shown in **Table 2**. These bridge piers are designed on the type II ground by Design Specifications of Highway Bridges [Japan Road Association, 1996]. The steel piers are stiffening box sections and RC piers are square sections. The width-thickness parameter and the slenderness parameter of steel piers are set 0.3 to 0.6 and 0.25 to 0.65, respectively. Natural periods of steel and RC piers are from 0.38 to 1.41 second and 0.38 to 0.71 second, respectively. The width-thickness parameter and the slenderness parameter are defined as follows:

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

$$\bar{\lambda} = \frac{2h}{r}\cdot\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}} \tag{3}$$

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, n: Number of sub panel girded by stiffening member,

h: length of member, r: radius of gyration

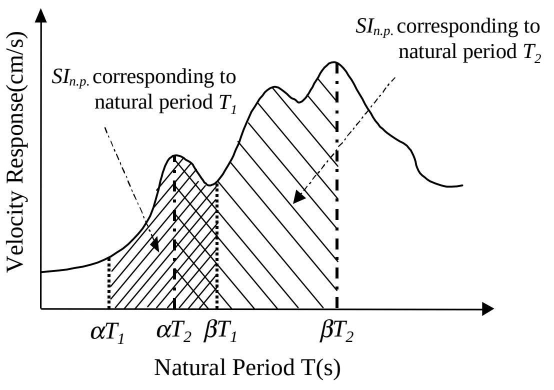


Figure 1: Conceptual Diagram of Proposed $SI_{n.p.}$

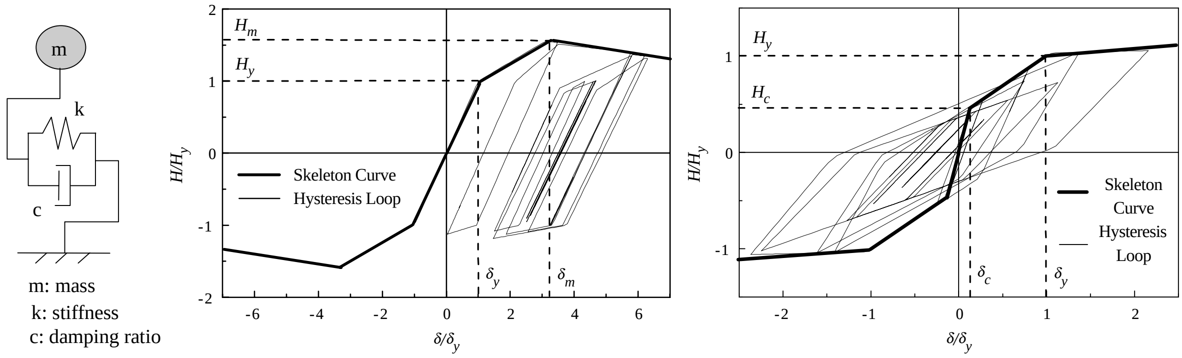
Table 2: Target bridge piers

Model Name	Type	width-thickness parameter	slenderness parameter	natural period
S3025	steel	0.30	0.25	0.38
S3045	steel	0.30	0.45	0.69
S3065	steel	0.30	0.65	1.02
S4525	steel	0.45	0.25	0.47
S4545	steel	0.45	0.45	0.86
S4565	steel	0.45	0.65	1.26
S6025	steel	0.60	0.25	0.53
S6045	steel	0.60	0.45	0.96
S6065	steel	0.60	0.65	1.41
RC38	RC	-	-	0.38
RC41	RC	-	-	0.44
RC56	RC	-	-	0.56
RC61	RC	-	-	0.61
RC71	RC	-	-	0.71

The single degree-of-freedom model is adopted as shown in **Fig. 2**(a) because the target structures are single column type bridge piers. The bases of models are fixed in the analysis. As the hysteresis characteristics of steel piers, a 2-parameter model proposed by Suzuki et al. is used [Suzuki et al., 1996]. This model can be presented the degrading of stiffness and strength with local buckling of plates. The skeleton curve is assumed as tri-linear model as shown in **Fig. 2**(b). In **Fig. 2**(b), the example of 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**(b) indicate maximum horizontal force and maximum horizontal displacement, respectively. These parameters are decided by the estimation equations of the reference [Suzuki et al., 1996].

The degrading-tri-linear model (Takeda Model) as shown in **Fig. 2**(c), which can represent the degrading of unloading stiffness, is used as the hysteresis characteristics of RC piers. In **Fig. 2**(c), the example of hysteresis loop of RC44 is presented. H_c, δ_c, H_y and δ_y indicate horizontal force at initial crack, horizontal displacement at initial crack, yield horizontal force and yield horizontal displacement, respectively. These parameters are calculated based on the reference [Japan Road Association. 1997].

By comparing **Fig. 2**(b) to **Fig. 2**(c), it is found that steel piers behavior elastic until yield horizontal force but the stiffness of RC piers is degraded due to the initial crack before the yield horizontal force. Moreover, it represents that the unloading stiffness of steel piers is equal to the elastic stiffness but one of RC piers is smaller than the elastic stiffness. The difference of these hysteresis characteristics offers the key to understand the problem to be discussed in the next chapter.



(a) Spring-mass model (b) Hysteresis characteristic (steel) (c) Hysteresis characteristic (RC)

Figure 2: Analysis model

The damping ratio is 5% at the damping term of dynamic equation. Linear acceleration method is used in time history analysis. The peak ground velocities of all input seismic waves, which are described in **2.1** and the total number is eighteen, are set 25, 35, 50 and 75cm/s. Thus, nonlinear analyses are performed seventy-two times against one analysis model. The relevance of responses and indices of seismic motions is discussed using seventy-two results of dynamic analyses against each pier.

ANALYSIS RESULTS AND DISCUSSION

Calculation of Correlation between Dynamic Response and Index of Earthquake Motion

It is assumed that the distributions of the dynamic responses and the indices of seismic motions are logarithmic normal distribution because the dynamic response values and the indices of seismic motions are not negative values. With this assumption, the correlation is calculated by Eq. (4):

$$\log R = a + b \cdot \log I \tag{4}$$

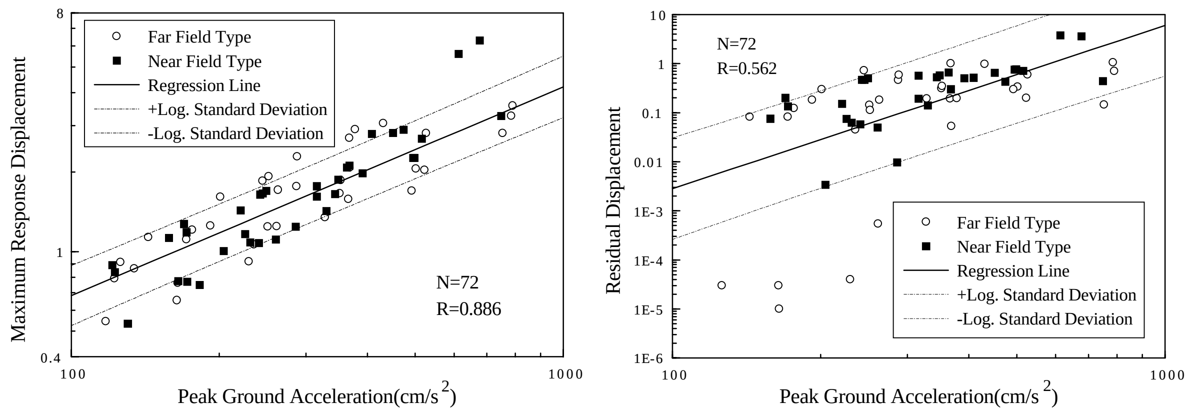
where, R: dynamic response value, I: index of seismic motions, a,b: regression coefficient

The result of S6025 is shown in **Fig. 3**. **Figure 3**(a) shows the relationship between the maximum response displacement and the PGA and **Fig. 3**(b) indicates the relationship between the residual displacement and the PGA. In these figures, the solid line presents the regression line and the dash-dotted lines represent the logarithmic standard deviations. The symbols of circle and square denote the results of near-field type earthquake and far-field type earthquake, respectively. N shows the number of analysis case and R shows the correlation coefficient.

As far as maximum displacement is concerned, **Fig. 3**(a) indicates that there are no significant difference between the results of near-field type and far-field type. Concerning residual displacement, **Fig. 3**(b) shows that there is not large difference between the results of near-field and far-field in the residual displacement range over 10^{-3}. Thus, in the further discussions, all the seismic waves are not separated into near-field type and far-field type and are treated as a population.

The correlation between the maximum response displacement and the PGA is strong such as its coefficient is 0.886. The scatter diagram (**Fig. 3**(a)) also shows that there is obviously the linear relationship between maximum response displacement and the PGA. However, it is not clearly if there is the linear relationship between the residual displacement and the PGA. In particular, the variations of residual displacement are very large in the acceleration range up to almost 250 cm/s^2. It would be better to say that the relationship between the residual displacement and the PGA is not explained by Eq. (4).

Against all the target piers, the correlation between maximum response displacement and the another indices of seismic motions is calculated by Eq. (4). In such cases, it is verified that the regression results are fit under significant level 0.01(1%) using t-distribution.

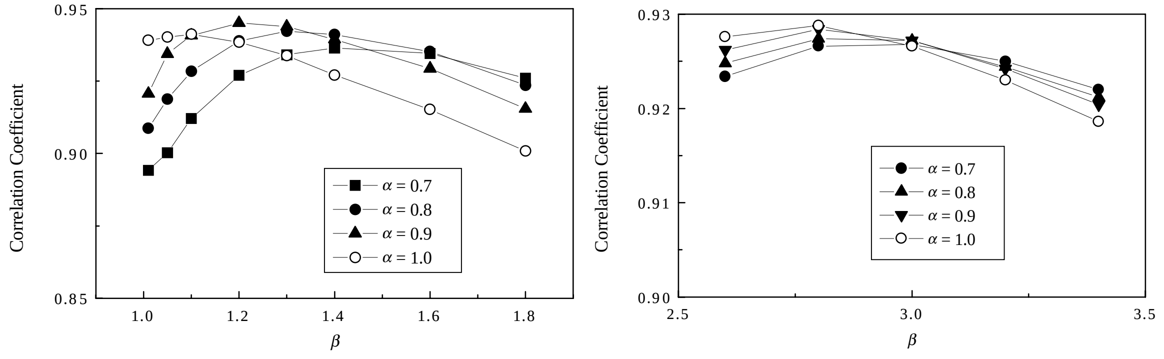


(a) Maximum displacement and PGA (b) Residual displacement and PGA

Figure 3: Scatter diagrams

Variation of Correlation due to natural period

The optimum parameters of α and β, which indicate integration ranges of natural-period-dependent SI shown in Eq. (1), are investigated. **Figure 4** indicates the results in the case of $\alpha \le 1.0$ and $\beta \ge 1.0$. **Figure 4**(a) shows the results of steel piers and **Fig. 4**(b) shows ones of RC piers. In these figures, the vertical axes show the mean values of correlation coefficient between maximum response displacement and natural-period-dependent SI and the horizontal axes show the parameter β. The square, closed circle, triangle and open circle denote the results of α = 0.7, 0.8, 0.9 and 1.0, respectively.



(a) Steel pier (b) RC pier

Figure 4: Relationship between α, β and correlation coefficient

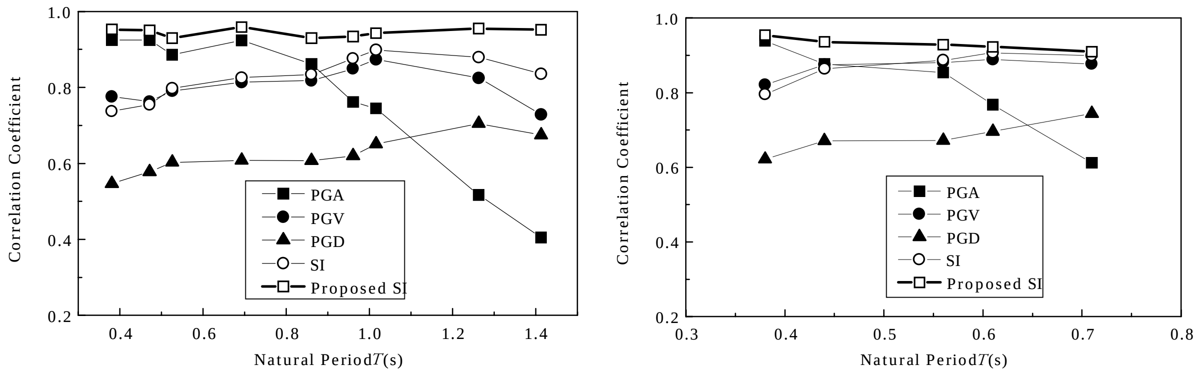
The values of α and β at the point the correlation coefficient is maximum can be regarded as the optimum parameters. For **Fig. 4**, the optimum parameters are α=0.9 and β=1.2 for steel piers, and α=1.0 and β=2.8 for RC piers. The optimum integration ranges are significantly difference between steel piers and RC piers. The reason for the difference of optimum integration ranges is the difference of prolongation of natural period due to degrading stiffness with damages. Moreover, the difference of prolongation of natural period comes from the hysteresis characteristics described in **2.3**. That is to say, the prolongation of natural period is short in the case of steel piers and is long in the case of RC piers.

Judging from the above, the optimum integration ranges which is used for proposed natural-period-dependent SI is decided as **Table 3**. However, the optimum integration ranges decided here is effectively only for bridge piers of single column which are predominated by the first-order mode. For other bridge types, which are predominated by highly-order modes, it might be necessary to investigate against each bridge type.

Table 2: Optimum integration ranges

Bridge Pier Type	α	β
Steel Pier	0.9	1.2
RC Pier	1.0	2.8

Figure 5 shows the correlation coefficient between the maximum response displacement and the index of seismic motions regarding all analysis models. **Figure 5**(a) shows the results of steel piers and **Fig. 5**(b) shows ones of RC piers. In these figures, the symbols of closed square, closed circle, triangle, opened circle and opened square denote the correlation coefficient of the PGA, PGV, PGD, SI and natural-period-dependent SI, respectively.



(a) Steel pier (b) RC pier

Figure 5: Correlation coefficient against maximum response displacement

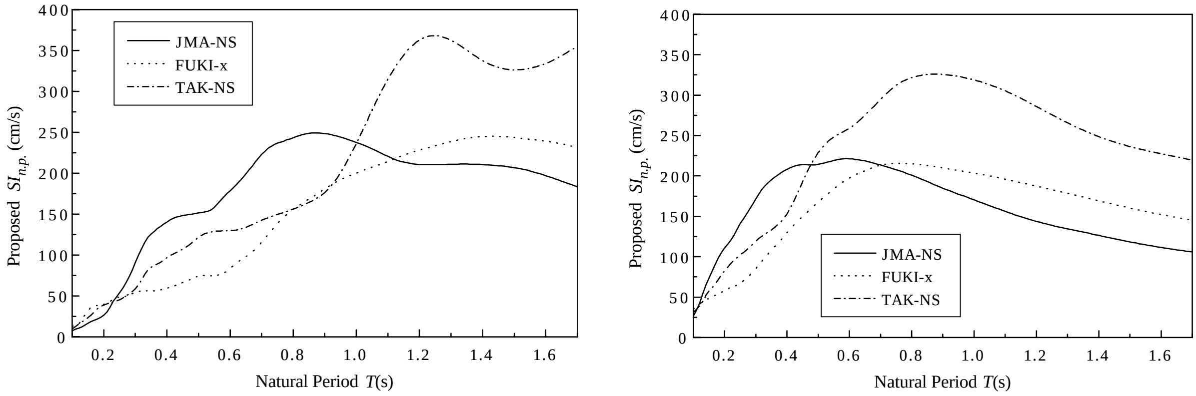
Figure 5 indicates that the correlation between the PGA and the maximum response displacement is strong in the natural period range up to 0.9 second in steel piers and up to 0.45 second in RC piers. However, the correlation decreases in reverse proportion to the natural period over 0.9 and 0.45 second, respectively. The difference of period between steel and RC piers comes from the difference of prolongation of natural period in the nonlinear region of piers. **Figure 5** also shows that the correlation coefficient of the PGV and the maximum response displacement is from 0.7 to 0.9 over a wide range of natural period. The relevance of the SI and maximum response is almost equal to one of the PGV and maximum response, though the SI is the index taking account of the periodic characteristics of seismic motions.

These results present that the correlation between the dynamic response and the basic index of seismic motions, such as PGA, PGV, PGD and SI, varies according to the natural period of bridge piers. In the other hand, **Fig. 5** indicates that the correlation coefficient between the maximum response displacement and natural-period-dependent SI is from 0.93 to 0.95 for steel piers and is from 0.91 to 0.95 for RC piers. Namely, it is clarified that the relevance of the maximum response displacement and natural-period-dependent SI is very strong for all the piers having different natural period. Therefore, it is fair to say that dynamic responses can be estimated adequately using natural-period-dependent SI as the index of seismic motions.

Natural-Period-Dependent SI Spectrum

Since the integration ranges of natural-period-dependent SI is changed according to the target structures, natural-period-dependent SI has the disadvantage of lacking the generality as a index of seismic motions. In the design, natural-period-dependent SI spectrum is here proposed in order to avoid the disadvantage. Namely, natural-period-dependent SI against many waves is calculated previously and is showed as spectrum. For instance, **Fig. 6** shows natural-period-dependent SI spectra against JMA-NS, FUKI-x and TAK-NS. **Figure 6**(a) indicates the spectra for steel piers and **Fig. 6**(b) indicates the spectra for RC piers. In these figures, the solid line, dotted line and dash-dotted line denote JMA-NS, FUKI-x and TAK-NS, respectively.

In a case of seismic design against a certain structure (natural period *T*), the severe waves for input earthquake waves used to dynamic analyses can be selected from natural-period-dependent SI spectra. In **Fig. 6**, it is found that JMA-NS and TAK-NS are the most severe waves for steel piers in the natural period range up to 1.0 second and over 1.0 second, respectively. **Figure 6** indicates that JMA-NS is the most severe wave for RC piers in the natural period range up to 0.4 second. As has been pointed before, the reason is the difference of prolongation of natural period due to degrading stiffness with damages.



(a) Steel pier (b) RC pier

Figure 6: Natural-period-dependent SI spectra

In order to compare with these spectra, **Fig. 7** shows the velocity response spectra. In this figure, the solid line, dotted line and dash-dotted line denote JMA-NS, FUKI-x and TAK-NS, respectively. It is obvious that the response spectra estimate elastic dynamic response exactly. However, the response spectra can not estimate approximately in nonlinear response.

Figure 6(a) is similar to **Fig. 7** because the integration range of natural-period-dependent SI for steel piers is very close. Moreover, it is found that **Fig. 6**(b) is significantly difference from **Fig. 7** because of the wide integration range of natural-period-dependent SI for RC piers. For instance, the velocity response spectra indicate that the severe seismic wave for RC piers is TAK-NS in the natural period range down to 1.1 second. However, natural-period-dependent SI spectra show that the severe wave for RC piers is TAK-NS in the range down to 0.5 second. These results show that there is possibility to underestimate the nonlinear response with TAK-NS using the velocity response spectrum in the natural period range from 0.5 to 1.1 second.

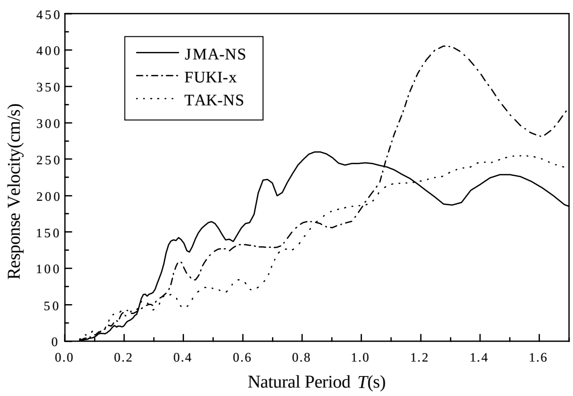


Figure 7: Velocity response spectra

CONCLUSIONS

Based on this study, following conclusions can be stated:

1. The correlation between peak ground acceleration and maximum response displacement is strong in the natural period range up to 0.9 second in steel piers and up to 0.45 second in RC piers. However, the correlation decreases in reverse proportion to the natural period over 0.9 and 0.45 second, respectively. The difference of period between steel and RC piers comes from the difference of prolongation of natural period due to degrading stiffness with damage.
2. The correlation coefficient of peak ground velocity and maximum response displacement is from 0.7 to 0.9 over a wide range of natural period. The relevance of spectrum intensity and maximum response is almost equal to one of peak ground velocity and maximum response, though spectrum intensity is the index taking account of the periodic characteristics of seismic motions.
3. Natural-period-dependent SI, which is calculated by the integration of the velocity response spectrum in the range corresponding to the natural period of the target piers, is proposed. The optimum integration ranges for natural-period-dependent SI are from $0.9T$ to $1.2T$ for steel piers and from $1.0T$ to $2.8T$ for RC piers, respectively. The difference of optimum integration ranges between steel and RC piers arises from the difference of prolongation of natural period.
4. For both steel and RC piers, the correlation between dynamic maximum displacement and natural-period-dependent SI is very strong over a wide range of natural period. The severe seismic waves against target structures can be selected by natural-period-dependent SI spectrum.

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