A STUDY ON FORCE-DISPLACEMENT-BASED SEISMIC DESIGN OF SINGLE-COLUMN RC PIERS

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As a fundamental study to develop a force-displacement-based seismic design of RC piers, the acceleration-displacement response spectrum (ADRS) of SDOF systems with elastic-plastic bilinear restoring force characteristics is obtained by inelastic response analysis, provided that the value of modified Park-Ang damage index D of a SDOF system equals the required value $D_r=0.4$ that almost correspond to the repairable limit of the system. The regression equation of ADRS is derived and applied to a force-displacement-based seismic design of a single-column RC pier. The design results by the proposed method are compared with the ones by the ductility design method of Japanese Specifications.

Key Words: *acceleration-displacement response spectrum, modified Park-Ang damage index, force-displacement-based seismic design, single-column RC pier*

1. INTRODUCTION

Presently, the so called “force-based seismic design method”, as represented by the Japanese Design Specifications of Highway Bridges, Part V Seismic Design¹⁾ (hereafter referred to as the Japanese Specifications), is the main method of the seismic design of a structure. In the force-based design method, the design seismic force of a target structure is specified on the basis of the acceleration response spectrum. The structure is then designed to maintain its serviceability securely in the event of moderate ground motions with high probability of occurrence (hereafter referred to as the Level 1 earthquake motions), and to ensure the required lateral capacity against the extreme ground motions with low probability of occurrence (hereafter referred to as the Level 2 earthquake motions).

In the force-based seismic design method, however, there are some points that should be improved. For example, the design procedure to verify whether the response displacement ductility factor

of a designed structure is close to the allowable one has not been stipulated, despite the fact that the design seismic force of a target structure is calculated by using the allowable displacement ductility factor. Also it is well known that the force (stress) produced in a structure due to some loads is essentially dependent on the deformation (strain) of the structure. Consequently the degree of damage of a structure due to earthquake motion, that is, the various required performance levels of the structure, are described more easily and more clearly by the response value of deformation than that of force. Recently, therefore, a new seismic design method by the use of spectral displacement (deformation), which is more closely related to the seismic safety of the structure under severe earthquake motions, namely the displacement-based design method using the displacement response spectrum, has been proposed and examined^{2), 3), 4)}. However, the relation between the current force-based method and the new displacement-based method has yet to be made clear.

Under such a background, Fajfar^{5), 6)} has proposed a modified capacity spectrum method, originally proposed by Freeman et al, in order to more easily verify the safety of a structure subjected to severe earthquake motions by comparing the acceleration displacement response spectrum (hereafter referred to as the ADRS) obtained by inelastic response analysis, with the capacity spectrum of the structure. This method is considered to be useful to develop a more rational seismic design method of a structure in the future⁷⁾ because the method makes it possible to verify the seismic safety of a structure from both the force and the displacement.

In this study, therefore, as a fundamental study for applying this new seismic design method (verifying method of seismic safety) to a road bridge in Japan by use of the ADRS, the authors first simulate seventy-two earthquake motions so that the acceleration response spectra of the simulated motions agreed well with the ones for the Level 2 earthquake motions prescribed in the Japanese Specifications. The ADRS for bi-linear SDOF systems with elastic-perfectly plastic restoring force characteristics, which is used in the ductility design method of the Japanese Specifications, is obtained by the inelastic seismic response analysis inputting the simulated earthquake motions. Here the ADRS is a damage controlled spectrum because the ADRS is obtained with the condition that the value of modified Park-Ang damage index D of the SDOF system is equal to the required value, $D_r=0.4$ that almost correspond with the repairable limit of the SDOF system. Thus the regression equation of the ADRS is derived. The authors then proposed a seismic design method named force-displacement-based seismic design method of a single-column RC pier, by using the derived regression equation. The force-displacement-based seismic design method is one of the capacity spectrum methods proposed by Fajfar and by which the RC pier is designed so as to satisfy both its target strength and target displacement. And the design results for a single-column RC pier by the proposed method are compared with the ones by the current ductility design method of the Japanese Specifications. In consequence, it is found that the proposed method requires more ductility of the pier than the current method when the target strength of the proposed method and the design seismic force in the current ductility design method are the same and that, when the target response displacement of the both methods is almost the same, the proposed method requires more lateral capacity. Therefore the proposed

force-displacement-based seismic design method can be a safer seismic design method of the RC pier.

The outline of this study is as follows.

2. FORCE-DISPLACEMENT-BASED DESIGN

Here, the capacity spectrum method, which is the basis of the force-displacement-based seismic design method proposed in this study, is briefly explained. Then the ADRS and the capacity spectrum of a structure required for the capacity spectrum method are simply described.

(1) Capacity spectrum method

In the capacity spectrum method, the seismic safety for both the force and displacement of a structure are verified concurrently, by the graph transaction as shown in **Fig.1**, comparing the capacity (allowable) values of acceleration and displacement with their demand (response) values. The capacity spectrum in **Fig.1** is obtained by dividing the load (Q^*) of load-displacement (Q^*-D^*) relation by the equivalent mass m^* for the equivalent SDOF system of a target structure which will be described in **2.(3)**. That is, the capacity spectrum method is a design method to confirm that the lateral capacity (allowable acceleration S_a) of the designed structure exceeds the target strength (response acceleration S_a'), i.e., $S_a \geq S_a'$ and that the target (response) displacement (S_d) is almost equal to the allowable displacement (S_d'), i.e., $S_d \cong S_d'$, by using the ADRS corresponding to the input earthquake motion for design. Furthermore, the allowable acceleration S_a and displacement S_d' in **Fig.1** and in the text are obtained, respectively, by dividing the lateral capacity (Q^*) of the designed struc-

Fig.1 Capacity spectrum method

ture by equivalent mass (m^*), i.e., $S_a = Q^*/m^*$, and by multiplying the allowable displacement ductility factor μ_a by the yield displacement δ_y of the designed structure, i.e., $S_d = \mu_a \times \delta_y$. Also the S_a' and S_d indicate the response (target) acceleration and the response (target) displacement which are specified by the ADRS.

(2) ADRS (Acceleration displacement response spectrum)

The ADRS, which is the basis of the capacity spectrum method described in section (1), is the spectrum plotting the relation between the spectral acceleration S_a' on vertical axis and displacement S_d on horizontal axis. In this relation between the S_a' and S_d defined by equation (3), the natural period T of a structure can be obtained by equation (4) if the value of gradient θ of the line which connects the demand point of the structure and the origin is known, as illustrated in **Fig. 1**.

$$S_{de} = \frac{1}{\omega_0^2} S_{ae} = \frac{T^2}{4\pi^2} S_{ae} \quad (1)$$

$$S_a' = S_{ae} \cdot R_r \quad (2)$$

$$\begin{aligned} S_d &= \mu_d R_r S_{de} \\ &= \mu_d R_r \frac{T^2}{4\pi^2} S_{ae} = \mu_d \frac{T^2}{4\pi^2} S_a' \end{aligned} \quad (3)$$

$$\tan \theta = \frac{S_a'}{S_d} = \frac{1}{\mu_d} \left(\frac{2\pi}{T} \right)^2 \quad (4)$$

in which S_{ae} , S_{de} , S_a' and S_d are the response spectral values of the elastic acceleration, pseudo elastic displacement, inelastic acceleration and inelastic displacement, respectively, and ω_0 , R_r and μ_d are the natural circular frequency of the structure, required yield strength ratio (inverse number of the reduction factor) and displacement ductility factor, respectively.

Consequently, the ADRS can be obtained by the elastic acceleration response spectrum S_{ae} through the required yield strength ratio R_r (See 3. (2)) and the displacement ductility factor μ_d . As will be described in 4. (1), the Level 2 earthquake motions used in this study are simulated so as to conform their elastic acceleration response spectra to the standard ones for dynamic analysis of the Japanese Specifications (See **Fig.4**). In this study, therefore, the value of the standard acceleration response spectrum of the Japanese Specifications is directly

Fig.2 Multi degree of freedom system and equivalent single degree of freedom system

used for elastic spectral response acceleration S_{ae} in equations (1) ~ (3) .

(3) Capacity spectrum ⁸⁾

The capacity spectrum in **Fig.1** required for the capacity spectrum method can be obtained by the following procedure.

- 1) First model the target structure into multi degree of freedom (MDOF) system as shown in **Fig.2**.
- 2) Next carry out the Push-Over (elastic-plastic) analysis of the MDOF system by the use of static load p_i ($i=1-n$) with constant load ratio which is properly assumed ⁹⁾ by the first mode shape, mass (m_i) ratio of the system or the like.
- 3) Obtain the relation between the total shearing force $V = \sum p_i$ and the displacement D_t of the characteristic point of the MDOF system by using the results of Push-Over analysis, and decide the deformation shape ratio Φ_i of the system (See **Fig.2**) corresponding to the representative point on the V - D_t curve (e.g. at the yield point of the MDOF system).
- 4) Obtain the mass m^* and load Q^* - displacement D^* relation of the equivalent SDOF system by the use of Φ_i , D_t , m_i and p_i .
- 5) Finally, linearly approximate the obtained Q^* - D^* relation adequately, then obtain the capacity spectrum by dividing the load Q^* of the linearly approximated Q^* - D^* curve by the mass m^* .

In this study, however, the above-mentioned transformation is not employed, since the single-column RC pier that is chosen as the target of this study is modeled to the SDOF system in the Japanese Specifications. Therefore the capacity spectrum is obtained directly, by dividing the load of the load-displacement relation, obtained by the Push- Over analysis of the ductility design method in the Japanese Specifications, by the equivalent

mass.

3. DAMAGE INDEX D AND REQUIRED YIELD STRENGTH RATIO R_r

(1) Damage index D

In recent years, various indexes for evaluating the seismic damage of a structure quantitatively, that is, the damage indexes have been proposed. In this study Park-Ang damage index $D^{10)}$, which is often quoted due to the fact that the index is based on many experiment results and the fact that the mathematical expression is simple and manageable, is modified so that the value of D may be negative under elastic response, and may be positive only under inelastic response as in equation (5). Then the damage controlled ADRS, described in the next 3. (2), is obtained by using the modified index (hereafter referred to as the modified Park-Ang damage index).

$$D = \frac{(\mu_d - 1) + \beta \cdot \mu_h}{\mu_u - 1} \quad (5)$$

in which β is a positive coefficient which depends on the cross section characteristics of a RC member, and μ_d , μ_h and μ_u are the displacement ductility factor, energy ductility factor and ductility capacity defined by the following equation, respectively.

$$\mu_d = x_{\max} / x_y, \mu_h = E_h / (Q_y \cdot x_y), \mu_u = x_u / x_y \quad (6)$$

in which $x_{\max}$, x_y and x_u in equation (6) are the maximum response displacements, yield displacement and ultimate displacement, respectively. E_h and Q_y are the absorbed hysteretic energy and yield strength.

(2) Required yield strength ratio R_r

One of the important parameters which control the damage of a structure under severe earthquake motions is the yield strength ratio $R^{11), 12)}$ defined by equation (7).

$$R = Q_y / Q_{e \max} = x_y / x_{e \max} \quad (7)$$

in which $Q_{e \max}$ and $x_{e \max}$ are the maximum response restoring force and maximum response displacement when regarding the structure as an elastic sys-

tem.

In this study, the value of R by which the value of damage index D in equation (5) becomes equal to its required value D_r (the value which specifies a certain damage condition of the structure) is defined as the required yield strength ratio $R_r^{12)}$ for the D_r , and the R_r value is obtained by repeating the inelastic energy response analysis in which the value of R is fluctuating. Then the spectral value of acceleration response S_a' is calculated by substituting the standard spectral value of acceleration response S_{ae} of the Japanese Specifications, as above-mentioned, and the value of R_r into equation (2). Also the spectral value of displacement response S_d is calculated by substituting the calculated S_a' and the displacement ductility factor μ_d which correspond to the R_r into equation (3). Therefore, the ADRS obtained in this study is damage controlled spectrum such that the degree of damage of the SDOF structure under Level 2 earthquake motions is equal to the damage condition specified by the required value D_r of the modified Park-Ang damage index D .

4. INPUT EARTHQUAKE MOTION AND STRUCTURAL CHARACTERISTICS

(1) Input earthquake motion

The seismic damage of a structure is affected not only by its structural characteristics but also by the characteristics of an input earthquake motion. In this study, therefore, in order to maintain the generality of analytic results, 72 simulated earthquake motions (acceleration waves) are generated^{13), 14)} so as to conform their elastic acceleration response spectra to the standard ones of the Level 2 earthquake motions for dynamic analysis specified in the Japanese Specifications¹⁾, and the 72 acceleration waves are used as the input earthquake motions. On that occasion, for each soil condition of soft soil, moderate soil and stiff soil (hereafter referred to as the Group I soil, Group II soil and Group III soil, respectively), the authors simulated every 12 wave for three different magnitude $M=8.0$, 8.2 and 8.4 with the epicenter distance Δ of 100km, 150km, 200km and 250km in plate boundary earthquakes (hereafter referred to as the Type I earthquakes) and for three different magnitude $M=7.0$, 7.2 and 7.4 with the epicenter distance Δ of 5km, 10km, 20km and 30km in inland earthquakes (hereafter referred to as the Type II earthquakes). In a time history response analysis, however, the duration of these simulated earthquake motions is unified to be 81.92

seconds including the part with strong motion. An example of the acceleration waveform of the simulated earthquake motions is shown in **Fig.3**, and the comparison between the targeted acceleration response spectra of the Japanese Specifications and those of simulated earthquake motions each of which correspond to the targeted one is shown in **Fig.4**, respectively.

(2) Structural characteristics

In this study, as a fundamental research, elastic perfectly plastic bi-linear model with plastic rigidity ratio $\gamma=0.0$ is used as the restoring force characteristics of the SDOF structure, following the ductility design method for the single-column RC piers in the Japanese Specifications. As for the dumping factor h , natural period T and ductility capacity μ_u of the structure, the values shown in **Table 1** are employed. Also the value of $D_r=0.4$, as shown in **Table 1**, is adopted as the required value of the damage index D in equation (5), and the hysteretic damage of the structure is considered by the use of mean value of the positive coefficient $\beta=0.15$ in equation (5).

In addition, the relation between the value of damage index D and the damage condition of a structure is not necessarily clear. In this study, however, the authors regard $D_r=0.4$ which is close to the repairable limit 0.36, for the time being, as the equivalent of the damage condition (repairable damage) of the important bridges (hereafter referred to as the Type B bridges) for the Level 2 earthquake motions in the Japanese Specifications, by referring to the relation of Geobarah et al. as shown in **Table 2**.

(3) Analytic method

In this study, the above-mentioned required yield strength ratio R_r (See **3. (2)**) of all the combinations of every input earthquake motion described in section (1) and every value of structural characteristics

Fig.4 Acceleration response spectra

Table 1 Structural characteristics

Dumping factor	h	0.05
Plastic rigidity ratio	γ	0.00
Natural period	T	30 values within the range of 0.1 to 3.0 seconds changing by 0.1 second
Ductility capacity	μ_u	1.00 (elastic response) 1.5, 2.0, 3.0, 4.0, 6.0, 8.0
Required value of Park-Ang damage index D	D_r	0.40
Positive coefficient	β	0.15

Table 2 Mutual relation between damage condition and modified Park-Ang damage index D

Damage condition	Damage index
Elasticity limit	0.08
Minor damage limit	0.18
Repairable limit	0.36
Collapse	0.60

stated in section (2), is obtained by inelastic energy response analysis. On that occasion the well-known equation of motion and energy response equation as shown in equations (8) and (9) are used, and the natural circular frequency $\omega_0 = (k_0/m)^{1/2}$, (here the k_0 is the elastic rigidity of the bi-linear model), is

Fig.3 Acceleration waveform of simulated earthquake motion

kept constant during the hysteretic vibration. The equation of motion in (8) is solved by the use of linear acceleration method with the constant time increment $dt=0.005$ seconds. Then the displacement ductility factor $\mu_d = x_{\max}/x_y$ is calculated from the obtained maximum response displacement $x_{\max}$. Also the absorbed hysteretic energy $E_h = \int Q(x) dx$ in equation (9) is calculated by numerical integration, making use of the response value of the each time step which is obtained by solving the equation (8), and after calculating the energy ductility factor $\mu_h = E_h/(Q_y \cdot x_y)$ the value of damage index D is obtained by equation (5).

$$m\ddot{x} + c\dot{x} + Q(x) = -m\ddot{x}_0 \quad (8)$$

$$m \int \ddot{x} dx + c \int \dot{x} dx + \int Q(x) dx = -m \int \ddot{x}_0 dx \quad (9)$$

in which $\ddot{x}$, $\dot{x}$ and x are relative acceleration, velocity and displacement, respectively, and $\ddot{x}_0$, m and $Q(x)$ are acceleration of input earthquake motion, mass, and restoring force, respectively. And $c (=2h\omega_0 m)$ is dumping coefficient.

5. ADRS AND ITS REGRESSION EQUATION

In order to carry out the seismic design of a structure by using the force-displacement-based seismic design method described in chapter 2., it is necessary to prepare the regression equation for the ADRS. In this study, therefore, the ADRS for all combinations of each input earthquake motion and each value of structural characteristics of elastic perfectly plastic bi-linear model described in 4. (1) and (2) is obtained by inelastic energy response analysis, and the mean spectral value and standard deviation σ for every 12 simulated earthquake motion mentioned above, is calculated. Then the regression equation for the spectrum obtained by adding the standard deviation σ to the mean spectrum (hereafter referred to as the (mean+ σ) spectrum) is derived. In this chapter, the analytic results relating to the derivation of the regression equation, derived regression equation and the (mean+ σ) spectrum are described and illustrated.

(1) Comparison of analytic result

Fig.5 shows the comparison of the 12 ADRS (the thin lines) of the elastic perfectly plastic bi-linear model, for every 12 simulated earthquake motion of

each earthquake Type and soil Group, together with their mean spectrum (the broken line) and the (mean+ σ) spectrum (the thick line) in the case of dumping factor $h=5\%$, value of the required damage index $D_r=0.4$, positive coefficient $\beta=0.15$ and ductility capacity $\mu_u=4.0$. In addition, the marks $\circ$, $\triangle$, $\square$ and ∇ plotted on the (mean+ σ) spectrum in the figure indicate each demand (spectral acceleration and displacement) which correspond to the natural period $T=0.5, 1.0, 1.5$ and 2.0 seconds, respectively, for reference. It is written also that the coefficient of variation for the mean value of every 12 simulated earthquake motion is only 0.2 (most are 0.1) at the most, and it is relatively small, though the coefficient varies to some extent depending on the ductility capacity μ_u , earthquake Type and soil Group.

It is found in Fig.5 that the two or three spectra among the ones of 12 simulated earthquake motions take larger spectral value than that of the (mean+ σ) spectrum for whole natural period range considered in this analysis. Notably the values of some spectra are considerably higher than the value of the (mean+ σ) spectrum in the range of the natural period $T=1.0$ (mark $\triangle$)~ 1.5 seconds (mark $\square$) of the soil Groups II and III for Type II earthquake motion. Therefore, the (mean+ σ) spectrum is not necessarily the safest spectrum. In this study, however, the regression equation for this (mean+ σ) spectrum is derived for the time being, and the spectrum obtained by the regression equation is used as the ADRS in the force-displacement-based seismic design method described in Chapter 2.

Next Fig.6 compares the effect of the difference in ductility capacity μ_u ($=1.0, 2.0, 4.0$ and 8.0) on the ADRS, i.e., on the (mean+ σ) spectrum in the case of each earthquake Type and soil Group, and the marks $\circ$, $\triangle$, $\square$ and ∇ in the figure indicate the demand corresponding to the natural period $T=0.5, 1.0, 1.5$ and 2.0 seconds, respectively, as same as in Fig. 5.

From Fig.6 it is found that the spectral value of acceleration becomes smaller as the value of ductility capacity μ_u gets larger, not regarding the earthquake Type and soil Group. This is due to the fact that the value of required yield strength at which the damage of the structure reaches a certain limit state (here the state of required value of damage index $D_r=0.4$) remains small in the structure with a larger value of μ_u and higher ductility compared with the one which has a smaller value of the μ_u . Meanwhile the spectral value of displacement varies depending on the earthquake Type and soil Group and the value of ductility capacity μ_u . Namely the spectral

Fig.5 Comparison of ADRS for each 12 wave and for the (mean+ σ)ADRS

value in Type I earthquake motion increases as the value of the μ_u becomes greater up to the natural period $T=1.0$ second (up to the marks $\circ$ and $\triangle$ in the figure) in the case of soil Groups I and II, and up to 1.5 seconds (up to the mark $\square$) in soil Group

. As to Type II earthquake motion, in this connection, the range of the natural period T in which the similar tendency is shown gets up to 0.5 seconds (up to the mark $\circ$) narrower in soil Groups I and II, and up to 1.0 second (up to the mark $\triangle$) narrower in soil Group III. However, such a small and large constant relation of spectral displacement due to the value of the μ_u gradually disappears in the natural period range over the above-mentioned range both in Type I and Type II earthquake motions. Namely the form of the ADRS becomes complex when $T \geq 1.5 \sim 2.0$ seconds in Type I earthquake motion, and when $T \geq$ about 1.0~1.5 seconds in Type II earthquake motion, though the range of T depends on the soil Group.

Next in Type I earthquake motion the spectral value of acceleration increases almost evenly as the

soil condition becomes softer from Group I to Group III for all the ductility capacity μ_u as same as the standard acceleration response spectra S_{ae} of the Japanese Specifications (See Fig.4) to which the acceleration response spectra of simulated earthquake motions are conformed. On the contrary, except for the range of $T=0.5 \sim 1.5$ seconds, the spectral value in Type II earthquake motion decreases in a period range shorter than the excepted one ($T=0.5 \sim 1.5$ seconds) as the soil condition gets softer. On the other hand the spectral value tends to increase in the range of T longer than the above-excepted range. Also the spectral value of displacement in Type I earthquake motion increases as the soil condition becomes softer in the whole natural period range considered in this analysis, regardless of the value of ductility capacity μ_u . Such tendency is more remarkable as the natural period T becomes longer. Also the similar tendency as in Type I earthquake motion is found even in Type II earthquake motion when the natural period $T \geq 1.0 \sim 1.5$ seconds, though the adverse result is

Fig.6 Comparison of effect of difference in ductility capacity μ_u on the (mean+ σ) ADRS

seen when $T \leq 0.5$ seconds. And when the natural period $T \leq$ about 1.5 seconds the spectral value of displacement in Type II earthquake motion becomes larger compared with the one in Type I earthquake motion. However, such a small and large relation of spectral displacement is not clear when $T \geq$ about 1.5 seconds.

(2) Derivation method of regression equation

The form of inelastic ADRS varies depending on the Type of earthquake motion, Group of soil condition and value of the ductility capacity μ_u as shown in Fig.6. Also the form becomes considerably complex when the natural period $T \geq 1.5 \sim 2.0$ seconds. Therefore, it is unfavorable to employ this ADRS directly as the spectrum for seismic design. On that account, in this study, the mean value of the required yield strength ratio R_r for every 12 simulated earthquake motion is first obtained considering the fact that the relation between above-mentioned equation (2) and equation (3), that is, the spectral value of the inelastic acceleration

response S_a' can be obtained if the required yield strength ratio R_r and the spectral value of the elastic acceleration response S_{ae} are known and the spectral value of the inelastic displacement response S_d can be obtained by the spectral values of the S_a' and the displacement ductility factor μ_d . And in order to derive the safety side regression equation of the ADRS for design, the regression equation of the (mean+ σ) spectrum for the R_r is derived after obtaining the (mean+ σ) value by adding the standard deviation σ to the mean value of the R_r . Then deriving the regression equation of the mean spectrum of displacement ductility factor μ_d , the ADRS for seismic design corresponding to the (mean+ σ) value is obtained by using these regression equations and the standard acceleration response spectrum S_{ae} of the Japanese Specifications (See equations (2) and (3)). The results derived are as follows:

a) Regression equation of R_r

As seen from equation (2), the required yield strength ratio R_r also varies depending on the

earthquake Type, soil Group and value of the ductility capacity μ_u as same as the ADRS in **Fig.6**. Therefore, in this study, the regression equation of the (mean+ σ) spectrum of the R_r for each combination of the earthquake Type and soil Group is derived as mentioned above. The results are as shown in equations (10) and (11).

[Type I earthquake motion]

$$\begin{aligned} R_r(T) &= f(\mu_u, T) \\ &= (C_0/\mu_u^2 + C_1/\mu_u + C_2) \\ &\quad + (C_3/\mu_u^2 + C_4/\mu_u + C_5) \cdot \ln(T) \end{aligned} \quad (10)$$

[Type II earthquake motion]

$$\begin{aligned} R_r(T) &= f(\mu_u, T) \\ &= (C_0/\mu_u^2 + C_1/\mu_u + C_2)/T^2 \\ &\quad + (C_3/\mu_u^2 + C_4/\mu_u + C_5)/T \\ &\quad + (C_6/\mu_u^2 + C_7/\mu_u + C_8) \end{aligned} \quad (11)$$

in which $C_0 \sim C_8$ in equations (10) and (11) are the regression coefficients for each earthquake Type and soil Group as shown in **Table 3**.

b) Regression equation of μ_d

As seen from equation (3), in order to obtain the regression equation of the inelastic displacement response spectra S_d , the regression equation of the displacement ductility factor μ_d which correspond to the R_r is required in addition to the inelastic acceleration response spectra S_a' or the required yield strength ratio R_r obtained in sub-section a) and the regression equation of the standard acceleration response spectra S_{ae} of the Japanese Specifications. In this study, therefore, the regression equation targeting the mean value of the μ_d for every 12 simulated earthquake motion is also derived as follows.

$$\begin{aligned} \mu_d(T) &= (E_0 \cdot \mu_u + E_1) \cdot T \\ &\quad + (E_2 \cdot \mu_u + E_3) \end{aligned} \quad (12)$$

in which the regression coefficients $E_0 \sim E_3$ in equation (12) are as in **Table 4**, respectively, corresponding to each combination of earthquake Type and soil Group.

c) Comparison of regression and analytic results

Fig.7 shows the comparison of regression result of the (mean+ σ) spectrum of the ADRS and the analytic one of the corresponding spectrum to the

Table 3 Regression coefficients $C_0 \sim C_8$ of R_r (Equations (10), (11))

	Type I			Type II		
	Group I	Group II	Group III	Group I	Group II	Group III
C_0	-0.686	-0.649	-0.590	0.001	-0.007	-0.011
C_1	1.403	1.344	1.267	0.002	0.016	0.021
C_2	0.275	0.296	0.317	-0.002	-0.008	-0.010
C_3	-0.199	-0.220	-0.241	0.075	0.160	0.199
C_4	0.310	0.340	0.386	-0.157	-0.325	-0.386
C_5	-0.114	-0.124	-0.149	0.081	0.161	0.187
C_6	—	—	—	-0.916	-0.919	-0.933
C_7	—	—	—	1.855	1.917	1.939
C_8	—	—	—	0.056	0.002	-0.010

Table 4 Regression coefficients $E_0 \sim E_3$ of μ_d (Equation (12))

	Type I			Type II		
	Group I	Group II	Group III	Group I	Group II	Group III
E_0	0.002	0.000	-0.007	-0.007	-0.016	-0.025
E_1	0.006	0.008	0.020	0.020	0.022	0.040
E_2	0.006	0.265	0.280	0.264	0.288	0.299
E_3	0.780	0.770	0.744	0.774	0.743	0.726

regression result in the case of ductility capacity $\mu_u = 2.0$ and 8.0 for each earthquake Type and soil Group. Here the regression result is obtained by substituting the regression equation of R_r and μ_d derived in the above sub-sections a) and b) and the standard acceleration response spectrum S_{ae} of the Japanese Specifications into equations (2) and (3).

It is found from **Fig.7** that the regression result reproduces the analytic result relatively well except for the range of natural period $T \geq 1.5 \sim 2.0$ seconds in which the form of the ADRS becomes to be complicated.

6. APPLICATION TO A SINGLE-COLUMN RC PIER

Here, by using the regression equation of the ADRS derived in chapter 5., the design procedure of the capacity spectrum method targeting a single RC pier is indicated. In this method what is called force-displacement-based design method, it is required that the lateral capacity of the pier exceeds its target strength and the response displacement of the pier is almost equal to its target. And the results of the numerical calculations of each procedure in the design example of a single-column RC pier with the design condition, described in the next section (1), is shown. Then the final design results are tabulated and are discussed and compared with the results obtained by the ductility design method of the Japanese Specifications.

Fig.7 Comparison of regression result and analytic result

(1) Design conditions

The T-shaped single-column RC pier with circular cross section as shown in **Fig.8** is employed as the target pier for design, and it is assumed that its superstructure is a simple girder bridge loaded with B live load (live load for important bridges). It also sets the foundation for the cast-in-place RC pile and the soil condition to be Group II, and the weight of superstructure, pier height and cross sectional diameter are set at $W_u=4900(\text{kN})(500(\text{tf}))$, $h_p=6(\text{m})$ and $\phi=2.5(\text{m})$, respectively. Other shapes and sizes of the pier are shown in **Fig.8**. In addition, the strength and elastic modulus of concrete and reinforcing bar are set at normal values as below, and arrangement of the reinforcing bar is set to be double without a cutoff.

[Concrete]

Compressive strength $f'_c=2.058(\text{MP}_a):(210(\text{kgf}/\text{cm}^2))$

Elastic modulus $E_c=2.303(\text{GP}_a):(2.35 \times 10^5 (\text{kgf}/\text{cm}^2))$

[Reinforcing bar (SD295)]

Yield strength $f_y=29.40(\text{MP}_a):(3000(\text{kgf}/\text{cm}^2))$

Fig.8 Shapes and sizes of designed pier

Elastic modulus $E_s=20.58(\text{GP}_a):(2.1 \times 10^6 (\text{kgf}/\text{cm}^2))$

(2) Design procedure of force-displacement-based design

a) Design procedure

Corresponding to the procedure 1)~9) of the design calculation flow shown in **Fig.9**, the explanation of the design procedure of the force-displacement-

ment-based design becomes as follows:

- 1) First, set the equivalent mass m^* of the target pier (equivalent SDOF structure), target acceleration S_a (target strength $F_y = m^* \cdot S_a$) and target displacement S_d etc.
- 2) Next, assume the ductility capacity μ_{u1} of the pier and describe the ADRS corresponding to this μ_{u1} (See 5. (2))
- 3) Plot the target value (S_a and S_d) set in procedure 1) on this spectrum and calculate the gradient θ_1 of the line connecting the point (S_a and S_d) to the origin from equation (4). Next calculate the elastic natural period T_1 which satisfies equations (4) and (12).
- 4) Obtain the required yield strength ratio R_{r1} by substituting the μ_{u1} assumed in procedure 2) and T_1 obtained in procedure 3) into equations (10) or (11) depending on the Type of earthquake motion, and calculate the response acceleration value S_{a1} by substituting the obtained R_{r1} and the standard elastic response acceleration $S_{ae}(T_1)$ of the Japanese Specifications into equation (2). Then calculate the convergence determination error ε_{Sai} between this calculated S_{a1} and the target S_a from equation (13), and proceed to procedure 5) if the error is within the allowable range (e.g. $\varepsilon_{Sai} \leq 1\%$). Meanwhile, if the error is not within the range, move to procedure 6).

$$\varepsilon_{Sai} = \frac{|S_a - S_{ai}|}{S_a} \times 100(\%) \quad (i = 1, 2, \dots) \quad (13)$$

in which ε_{Sai} is the convergence determination error of the response acceleration value S_{ai} of i time repetition.

- 5) In the same way, calculate the maximum response value of displacement S_{d1} from equation (3), and obtain the convergence determination error ε_{Sd1} between this S_{d1} and the target S_d from equation (14). Then if the obtained ε_{Sd1} is within the range of allowable error (e.g. $\varepsilon_{Sdi} \leq 1\%$), move to procedure 7). If not, proceed to procedure 6).

$$\varepsilon_{Sdi} = \frac{|S_d - S_{di}|}{S_d} \times 100(\%) \quad (i = 1, 2, \dots) \quad (14)$$

in which ε_{Sdi} is the convergence determination error of the response displacement S_{di} of i time repetition.

- 6) Calculate the ductility capacity μ_u' which satisfies the relation of equation (2) by substituting the

Fig.9 Design calculation flow

target acceleration S_a for the S_a' of equation (2) and by substituting the natural period T_1 calculated in procedure 3) into the regression equations of S_{ae} and R_r (equation (10) or (11)) in the right side of the same equation. Then return to procedure 2) by setting this μ_u' as the assumed value μ_{u2} for the second time.

- 7) As in the ductility design method of the Japanese Specifications the ductility capacity μ_u is obtained as well as the amount of reinforcement etc. by making a cross sectional design so that the lateral capacity F_y' of the pier is equal to or larger than the target strength F_y set in 1) (to be $F_y' \geq F_y$).
- 8) Obtain the convergence determination error ε_μ between the ductility capacity μ_{ui} assumed in procedure 2) and the value $\mu_u (= \mu_{ui}')$ designed in procedure 7) from equation (15). Then finish the design calculation if the error is within the range of the allowable error (e.g. $\varepsilon_\mu \leq 1\%$), regarding the ductility capacity of the designed pier reproduces the assumed value μ_{ui} in procedure 2), that is, the response displacement S_{di} of the designed pier becomes almost equal to the target displacement.

ment S_d ($S_d \approx S_d$). Meanwhile, proceed to procedure **9)** if the error ε_μ is not within the allowable range.

$$\varepsilon_\mu = \frac{|\mu_{ui} - \mu'_{ui}|}{\mu_{ui}} \times 100(\%) \quad (i = 1, 2, \dots) \quad (15)$$

in which ε_μ is the convergence determination error of the ductility capacity, and μ_{ui} and μ'_{ui} are i time ductility capacity of the pier assumed in procedure **2)** and the ductility capacity obtained by the design calculation in **7)**, respectively.

- 9)** Repeat the design calculation of the cross section, changing the amount of main and tai reinforcements, and decide the cross section (amount of the reinforcement) such that $F_y \geq F_y$ and the convergence determination error ε_μ in equation (15) is within the range of the allowable error.

b) Design example

In accordance with design procedures **1)~9)** described in sub-section **a)**, the design calculation for the single-column RC pier with the shape and sizes as shown in **Fig.8** and with the design conditions described in section **(1)** is carried out to satisfy the values of the target acceleration $S_a=800$ (gal) and displacement $S_d=15$ (cm) subjected to Type II earthquake motion whose effect on the damage of the pier dominates that of Type I earthquake motion. The numerical results in each of the above-mentioned design procedures for this example are shown as follows:

[The first time]

- 1) Equivalent mass $m^* = 582.3(\text{kN} \cdot \text{s}^2/\text{m})$
Target acceleration $S_a = 800$ (gal)
 $[F_y = 4658.5(\text{kN})]$
Target displacement $S_d = 15$ (cm)
- 2) Ductility capacity $\mu_{u1} = 6.5$
- 3) Elastic natural period $T_1 = 0.54$ (second)
- 4) Response acceleration $S_{a1} = 820.6$ (gal)
Convergence determination error
 $\varepsilon_{Sa1} = 2.57\% > 1.0\%$ OUT move to **6)**
- 6) Ductility capacity
 $\mu'_{u1} = 6.95$
Return to **2)** setting this μ'_{u1} as the assumed value μ_{u2} of the second time.

[The second time]

- 2) Ductility capacity $\mu_{u2} = 6.95$
- 3) Elastic natural period $T_2 = 0.52$ (second)
- 4) Response acceleration $S_{a2} = 807.1$ (gal)
Convergence determination error

$$\varepsilon_{Sa2} = 0.89\% < 1.0\% \quad \text{OK} \quad \text{move to } \mathbf{5)}$$

- 5) Response displacement $S_{d2} = 15.2$ (cm)
Convergence determination error

$$\varepsilon_{Sd2} = 1.04\% > 1.0\% \quad \text{OUT} \quad \text{move to } \mathbf{6)}$$

- 6) Ductility capacity

$$\mu_{u2}' = 7.12$$

Return to **2)** setting this μ_{u2}' as the assumed value μ_{u3} of the third time.

[The third time]

- 2) Ductility capacity $\mu_{u3} = 7.12$
- 3) Elastic natural period $T_3 = 0.52$ (second)
- 4) Response acceleration $S_{a3} = 803.1$ (gal)
Convergence determination error
 $\varepsilon_{Sa3} = 0.38\% < 1.0\%$ OK move to **5)**
- 5) Response displacement $S_{d3} = 15.1$ (cm)
Convergence determination error
 $\varepsilon_{Sd2} = 0.39\% < 1.0\%$ OK move to **7)**
- 7) Repeat the design calculation of the cross section and decide the amount and arrangement of main and tie reinforcements (See **Table 5**) so that the lateral capacity $P_a (= F_y')$ becomes equal to or larger than the target strength $F_y = 4658.5$ (kN).
Lateral capacity
 $P_a = 4669.2(\text{kN}) > F_y = 4658.5(\text{kN})$ OK
- 8) Confirm whether the error ε_μ in equation (15) between the ductility capacity $\mu_{u3}' = 7.11$ of the designed pier (See **Table 5**) and the assumed value $\mu_{u3} = 7.12$ of the third time in **2)** is within the range of the allowable error.
Convergence determination error
 $\varepsilon_\mu = 0.21\% < 1.0\%$ OK end of design

In addition, **Fig.10** shows the comparison of the ADRS corresponding to the ductility capacity $\mu_{u3} = 7.12$ assumed in the third repetition and the capacity spectrum of the pier. From this figure it is found that the designed pier almost reproduces the target acceleration [strength] $S_a=800$ (gal) [$F_y=4658.5$ (kN)] and the target displacement $S_d=15$ (cm).

c) Comparison with the ductility design method

As is well known, the ductility design method of Japanese Specifications is not the method to satisfy both targets of acceleration (strength of the pier) and displacement concurrently, unlike the force-displacement-based seismic design method described in sub-section **a)**. And the design example by the proposed method, presented in **b)**, is the result obtained by use of the design ADRS which satisfies the required value $D_r=0.4$ of the modified Park-Ang damage index D , regarding that this required value $D_r=0.4$ is equivalent to the recoverable damage for the Level 2 earthquake motion of Type

B bridges in the ductility design method of Japanese Specifications, for the time being, as described in 4. (2). In addition, the value $D_r=0.4$ used in this study is slightly different from the repairable limit 0.36 proposed by Geobarah et al. (See Table 2), also the value of Geobarah et al. is the one for the RC building structure and not the value for the RC pier targeted in this study. Therefore it is not necessarily said that there is a good corresponding relation between the degree of damage for $D_r=0.4$ and the degree of repairable damage for the Level 2 earthquake motion of Type B bridges in the ductility design method of Japanese Specifications. From these points above, it is a little unreasonable to compare the results by the above-mentioned force-displacement-based design method with the results obtained by the ductility design method of the Japanese Specifications. However, in order to examine the qualitative difference between the two design methods, the design of the pier by the ductility design method is attempted in these cases; i) the case giving $S_a=800$ (gal) [$F_y=4658.5$ (kN)] as the design acceleration (design strength) and ii) the case giving the allowable design displacement as $\delta_a=\mu_a\cdot\delta_y=15$ (cm), targeting the pier with the same design condition, shape and size as in the design example presented in sub-section b). Here, the case i) is employed to examine the difference between the results of both designs when the lateral capacity P_a of the ductility design method is almost equal to that of the force-displacement-based design method, and in case ii) where it is employed to examine the difference between the two results when the allowable displacement $\delta_a=\mu_a\cdot\delta_y$ of the former almost equals the target displacement S_d of the latter.

The comparison of the difference in the design results, that is, the difference in the lateral capacity, displacement, amount of reinforcement, etc., between the two design methods are shown in Fig.10 and Table 5, respectively.

As seen from the figure and table the ductility capacity μ_u decreases considerably in the ductility design method of Japanese Specifications compared with the force-displacement-based seismic design method in this study, and the allowable displacement $\delta_a=\mu_a\cdot\delta_y$ which is considered to be equivalent to the target (response) displacement S_d in this method also decreases slightly, when the lateral capacity P_a is almost the same. Relating to this, the amount of main reinforcement increases and the amount of the tie reinforcement decreases.

The above-mentioned result in which the allow-

Fig.10 Comparison of capacity spectra between force-displacement-based design method and ductility design method

able displacement δ_a etc. decrease in the ductility design method seems to be caused by the obtaining method of the design seismic force. That is, in the ductility design method, the equivalent lateral seismic coefficient k_{he} is calculated by the use of allowable displacement ductility factor μ_a and the equal energy assumption. Then the design seismic force is obtained by multiplying the seismic coefficient k_{he} by the equivalent weight W . Therefore, if the equivalent weight (mass) and the design seismic force are given initially as in the comparison here, the value of the allowable displacement ductility factor μ_a is inevitably decided by these values and the ductility capacity μ_u is also decided (through the safety factor).

Furthermore, when the displacement is almost the same, the lateral capacity P_a and the ductility capacity μ_u decrease and the amount of both the main and tie reinforcements also decrease in the ductility design method compared with the force-displacement-based design method. This is due to the obtaining method of the design seismic force, as mentioned above.

From the above-mentioned results, it is found that the force-displacement-based seismic design method proposed in this study requires more toughness (ductility) after yielding if the target strength (lateral capacity) is almost the same and requires more lateral capacity if the response displacement is almost the same, compared with the

Table 5 Comparison of design results between force-displacement-based design method and ductility design method

		Force - displacement-based design method	Ductility design method (same lateral capacity)	Ductility design method (same displacement)
Target value	Equivalent mass m^* (kN·s ² /m)	582.3	582.3	582.3
	Target acceleration S_a (gal)	800	800 ¹⁾	—
	Target strength F_y (kN)	4658.5	4658.5 ²⁾	—
	Target displacement S_d (cm)	15	—	15 ³⁾
	Ductility capacity (assumed value) μ_u	7.12	—	—
Design value	Ductility capacity μ_u	7.11	3.74	4.19
	Elastic natural period T^* (sec)	0.52	0.49	0.50
	Yield displacement δ_y (cm)	5.04	4.86	4.80
	Allowable displacement ductility factor $\mu_a = S_a / \delta_y$	2.98	2.83	3.13
	Lateral capacity P_a (kN)	4669.2	4662.6	4354.0
	Yield acceleration S_{ay} (gal)	801.8	800.7	747.7
	Allowable displacement (cm)	15.1	13.8	15.0
Amount and arrangement of main and tie reinforcements	Main	Diameter	D35	D35
		Pitch (mm)	80	77
		Number of the bar	80×2	75×2
		Concrete cover (mm)	115~225	115~230
		Main reinforcement ratio ρ_f (%)	3.12	3.24
	Tie	Diameter	D22	D16
		Pitch (mm)	101×2	101×2
		Tie reinforcement volume ratio ρ_s (%)	1.35	0.69
				0.76

1) Design (target) acceleration 2) Design (target) seismic force 3) Allowable design (target) displacement

ductility design method of the Japanese Specifications. It is said, therefore, that the proposed method in this study is a safer design compared with the ductility design method of the Japanese Specifications.

7. CONCLUSIONS

In this study, the acceleration displacement response spectrum (ADRS) used in the capacity spectrum method is obtained by the inelastic response analysis and after examining the analytic result the regression equation of the (mean+ σ) spectrum for the required yield strength ratio R_r and the mean spectrum of the displacement ductility factor μ_d are derived for each combination of the earthquake Type and soil Group. Then, the inelastic ADRS for design which satisfies the required value $D_r=0.4$ of the modified Park-Ang damage index D , by using these regression equations of the R_r and μ_d and the standard acceleration response spectra S_{ae} for the Level 2 earthquake motion of the Japanese Specifications is derived. And the capacity spectrum method, that is, the force- displacement-based

seismic design method for a single-column RC pier is proposed by use of the derived ADRS, and the qualitative difference in the design results, between the proposed method and the ductility design method of the Japanese Specifications, are compared and discussed.

The results obtained in this study are summarized as follows:

- 1) From the relation of equations (1)~(3), the regression equation of the ADRS for design is derived by use of the regression equations of the required yield strength ratio R_r , displacement ductility factor μ_d and standard acceleration response spectra S_{ae} of the Japanese Specifications. Then the desirable result of the ADRS with relatively high accuracy is obtained in the range of natural period $T \leq$ about 1.0 second which is supposed to correspond with the range of natural period T of a large number of single-column RC piers, though the accuracy decreases slightly in the longer range of natural period $T > 1.0 \sim 1.5$ seconds of the structure.
- 2) A single-column RC pier which satisfies the target strength $F_y=4658.5(\text{kN})[S_a=800(\text{gal})]$ and the

target displacement $S_d=15(\text{cm})$ is designed by the force-displacement-based seismic design method by use of the ADRS for design derived in this study. Comparing the design results with the ones by the ductility design method of the Japanese Specifications in which the design seismic force is set to be equal to the target strength F_y , the force-displacement-based seismic design method proposed in this study requires greater toughness (ductility) after yielding compared with the ductility design method due to the obtaining method of the design seismic force in the ductility design method described in 6. c). This result means that the proposed design method is the safer method compared with the ductility design method.

- 3) Similarly, the design by the ductility design method is carried out so that the allowable displacement $\delta_a=\mu_a\cdot\delta_y$ is equal to the target displacement $S_d=15(\text{cm})$ in the force-displacement-based seismic design method. Comparing the results with the ones by the force-displacement-based design of this study, the proposed method requires more lateral capacity and ductility capacity than the ones by the ductility design method due to the same reason as described above in 2). This also means the proposed method is the safer design.

In addition, the above-mentioned results in 2) and 3) are the ones in the case when the repairable damage in the ductility design method of the Japanese Specifications for Type B bridges is regarded to be almost equal to the damage corresponding to the value of the modified Park-Ang damage index $D=0.4$ as described above. Therefore, the design results by the proposed method could be different depending on the damage condition (seismic performance) of the structure to be assumed. Thus, the authors think it is necessary from now on to make similar comparisons and to make consideration as in this study which are based on the many design results obtained by use of the regression equation of the ADRS for the different required D_r values of the modified Park-Ang damage index D .

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