EFFECT OF FOUNDATION INTERACTION ON REQUIRED SEISMIC INTENSITY OF RC PIERS SUBJECTED TO LEVEL2 EARTHQUAKE MOTIONS

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SUMMARY

In this study, in order to investigate the effect of foundation-structure interaction on required seismic intensity k_r of RC piers, twenty-seven single column RC piers, varying in their heights, weights of a superstructure and pile foundations, are designed in accordance with current Japanese seismic design code for highway bridges. Each of the piers with pile supported footings is converted to S-R (sway-rocking) 3DOF analytical system with the restoring force characteristics of Q-hyst model for the pier and Hardin-Drnevich model for the foundation. Also twenty-one artificial earthquake motions, of which acceleration spectra coincide with the ones for level 2 earthquakes in the Japanese code, are employed. Then required seismic intensity k_{rI} for the S-R system and k_{rF} for R-B (rigid based) SDOF system are obtained from inelastic energy response analyses, provided that the value of modified Park-Ang's damage index D of the pier is equal to a designated value D_r (=0.4, 0.7 and 1.0). From the comparison of k_{rI} with k_{rF}, it is concluded that the consideration of foundation-structure interaction is essential to evaluate the seismic design force (intensity) /or damage of single column RC piers with pile foundations, especially for the pier on the site of soft soil and subjected to type Ⅱ earthquake.

1. INTRODUCTION

In order to develop a rational dual level (serviceability level and damage-control or survival level) /or performance based seismic design method, it is necessary to establish a reliable seismic design force and to evaluate the damage properly for a structure subjected to severe earthquake motions. Therefore extensive analytical and experimental studies on inelastic (energy) response of a structure excited by severe earthquake motions and inelastic hysteretic behavior of a member under cyclic loading have been carried out. From the studies, it has become common knowledge that the seismic design force for structures tolerating a certain degree of damage is defined by smoothed inelastic response spectra. And the inelastic spectra used in practice (seismic code) is obtained from elastic response spectra through the use of strength reduction factor R, corresponding to displacement ductility capacity or reduced capacity weighted with respect to anticipated cumulative damage [Krawinkler et al. 1992, Vidic et al. 1992]. As for the damage evaluation, Park-Ang's damage index D [Park et al. 1985] or its modified one [Kunnath et al. 1992] is widely used for reasons of its simplicity and broad experimental basis. There are a few studies [Toki et al. 1987,Yuan et al. 1993, Hirao et al. 1997, 1998] which discuss the effect of foundation-structure interaction on the damage or reduction factor of old RC piers [Kawashima et al. 1985]. However most of the previous studies are based on the assumption of rigid foundations. Therefore, how the foundation-structure interaction affects seismic intensity (force) /or damage of a structure has not been investigated enough, despite that the foundation of a structure is not generally rigid.

In this study, therefore, twenty-seven single column RC piers with pile foundations (pile supported footings) are designed in accordance with Japanese seismic design code [Japanese road association 1996]. Each of the piers is converted to S-R (sway-rocking) 3DOF system with the restoring force characteristics of Q-hyst model for the

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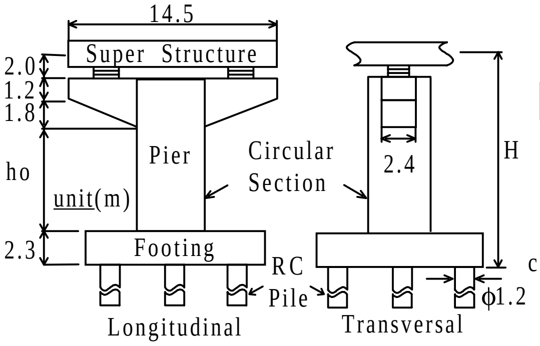
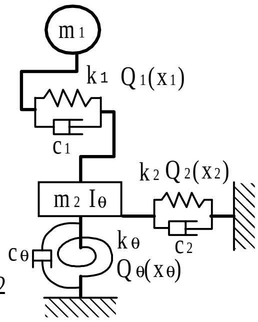
pier and





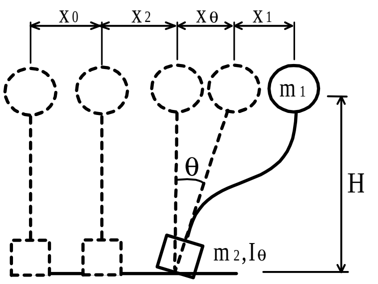


Fig.2: S-R system **Fig.3: Displacements of S-R system**

Fig.1: RC pier with pile foundation

Hardin-Drnevich model for the foundation, as an analytical model considering foundation-structure interaction. Twenty-one artificial ground motions [Hirao et al. 1997] are also employed as the level 2 input earthquake motions (extreme ground motions with low probability to occur) [Japanese road association 1996]. Then, required seismic intensity k_{rI} for the S-R system and k_{rF} for R-B (rigid based) SDOF system are obtained from inelastic energy response analyses, provided that the value of modified Park-Ang's damage index D of the pier is equal to a designated one D_r (=0.4, 0.7 and 1.0). After that, comparing the k_{rI} with k_{rF}, the effect of foundation interaction on the required seismic intensity (force) of RC piers is examined. Also, in order to discuss the suitableness of seismic design force in the Japanese code, equivalent seismic intensity k_{he} used for the design of RC piers is compared with the required ultimate seismic intensity k_{urI} in case of the designated damage Dr=0.4, being close to the repairable limit.

2. RC PIER AND ANALYTICAL MODEL

2.1 Single Column RC Pier

In this study, twenty-seven single column RC piers and their pile supported foundations as shown in Fig.1 are designed in accordance with Japanese seismic design code. On that occasion three different weights of superstructure (W_u= 4.90, 6.86 and 8.82 MN), heights of pier (h_0= 6, 10 and 14m; see Fig.1) and sandy soil conditions (N=30, layer thickness=10m, GCⅠ; N=20, layer thickness=20m, GCⅡ; and N=5, layer thickness=25m, GCⅢ; see Tables 1 and 2) are adopted. The structural parameters of the piers and foundations, required for the seismic response analyses (see **2.2** and **3.2**), are summarized in Table 1. The values of each parameter in Table 1 are obtained from the seismic design method, prescribed in the Japanese code. As for the meaning of parameters m_1, m_2, I_θ, k_1, k_2 and k_θ, refer to next section **2.2**.

2.2 Analytical Model

As an analytical model considering the foundation-structure interaction of a RC pier with pile supported footing, S-R 3DOF (sway-rocking three degrees of freedom) system shown in Fig.2 is employed. In this system, Q-hyst model and Hardin-Drnevich model are adopted as restoring force models of the pier ($Q_1(x_1)$) and sway-rocking foundation ($Q_2(x_2)$ and $Q_\theta(\theta)$), respectively. Also R-B SDOF (rigid based single degree of freedom) system is employed as an analytical model ignoring the foundation interaction. As for the damping factor h of the systems, the values of h=5% for all piers and h=10% for all foundations (sway and rocking dash-pots) are adopted, respectively. In Fig.2, m_1 is the mass of superstructure and half of pier; m_2 is the mass of footing and half of pier; I_θ is the moment of inertia of footing; k_1,k_2 and k_θ are the initial stiffness of pier, sway and rocking springs; and c_1,c_2 and c_θ are the viscous damping coefficients of pier, sway and rocking dash-pots, respectively. Displacements of the S-R 3DOF system in Fig.2 are defined as shown in Fig.3. Where x_0 is the ground displacement; x_1 is the relative displacement of pier. H is the distance between mass m_1 and rocking center (bottom center of footing); x_2 and θ are the sway displacement and rocking rotation of the foundation; and x_3 (=H θ) is the relative displacement caused by rocking of the foundation.
For reference, the relationship between ductility capacity μ_u and natural period T_{10} of the piers in Table 1 is illustrated in Fig.4, comparing the difference in earthquake types (typeⅠand typeⅡ) and soil conditions (GCⅠ, Ⅱand Ⅲ). The ratios of yield strength Q_{y2} for swaying and Q_{y3} (=$Q_{y\theta}$/H) for rocking of a pile foundation

to Q_{y1} of its pier, i.e., Q_{y2}/Q_{y1} and Q_{y3}/Q_{y1} are also shown in Fig.5, in the same manner as the μ_u in Fig.4. And Fig.6 illustrates the relationships among natural circular frequencies ω_1 for a pier, ω_2 for sway vibration and ω_θ for rocking vibration of its pile foundation.

Table 1: Structural parameters of RC pier and its S-R 3DOF system

	Wu (MN)	H (m)	m 1 (t)	k1 (kN/cm)	T_{10} (sec)	μ_u		γ		m 2 (t)	Iθ (t m²) *10⁶	k2 (kN/cm)	kθ (MN m)	Qy2/Qy1	Qy3/Qy1
						typeⅠ	typeⅡ	typeⅠ	typeⅡ						
N=30(hard @soil)	4.90	11	656	952	0.52	8.0	13.0	0.07	0.04	432	12.3	5335	13423	4.4	1.9
	6.86		864	1288	0.51	7.9	12.8	0.07	0.04	527	19.9	5623	17555	2.8	1.8
	8.82		1076	1656	0.51	8.1	13.1	0.07	0.04	649	36.2	6249	21815	2.8	1.8
	4.90	15	694	447	0.78	5.1	7.8	0.13	0.08	543	18.8	5628	17149	3.3	1.9
	6.86		907	600	0.77	5.1	7.8	0.12	0.07	690	36.9	6594	23628	3.3	1.8
	8.82		1128	839	0.73	5.3	8.2	0.12	0.07	736	46.4	8214	27071	3.0	1.8
	4.90	19	732	220	1.15	4.2	6.4	0.16	0.10	568	17.8	5595	16622	3.5	1.9
	6.86		959	314	1.10	4.4	6.6	0.15	0.09	665	31.0	7226	20993	5.2	1.9
	8.82		1179	406	1.07	4.5	6.8	0.14	0.09	745	39.7	7629	24798	3.1	1.8
N=20(moderate)	4.90	11	649	779	0.57	6.8	10.8	0.08	0.05	370	11.0	2335	7327	2.6	1.8
	6.86		856	1068	0.56	7.0	11.3	0.08	0.05	562	31.0	3966	17346	3.5	1.9
	8.82		1064	1371	0.55	7.2	11.7	0.08	0.05	590	33.7	4152	18448	2.6	1.8
	4.90	15	707	542	0.72	5.6	8.7	0.11	0.07	613	31.0	4099	17792	3.0	1.8
	6.86		928	752	0.70	5.9	9.2	0.10	0.06	694	39.7	4747	23826	3.0	1.8
	8.82		1143	954	0.69	6.0	9.5	0.10	0.06	649	31.0	5338	26470	3.0	1.8
	4.90	19	790	436	0.85	5.0	7.7	0.13	0.08	888	62.4	4958	32738	3.3	1.8
	6.86		1022	594	0.82	5.2	8.1	0.12	0.07	852	50.0	6734	38024	3.2	1.8
	8.82		1257	769	0.80	5.6	8.9	0.11	0.07	1185	114.0	8565	58523	3.0	1.8
N=5(soft @soil)	4.90	11	652	734	0.59	8.8	14.3	0.06	0.03	587	33.2	4803	23654	3.2	1.8
	6.86		864	1040	0.57	10.0	16.5	0.05	0.03	570	31.0	5283	26997	3.0	1.8
	8.82		1073	1335	0.56	10.3	17.0	0.05	0.03	974	107.0	6286	61373	3.3	1.8
	4.90	15	707	481	0.76	6.2	9.8	0.10	0.06	954	94.1	6232	58341	3.9	1.9
	6.86		936	693	0.73	7.5	12.1	0.07	0.04	852	56.2	8351	66579	3.8	1.8
	8.82		1151	878	0.72	7.7	12.5	0.07	0.04	998	94.1	11145	88815	4.8	1.9
	4.90	19	790	385	0.90	5.5	8.6	0.11	0.07	1140	103.9	8475	94779	4.1	1.9
	6.86		1033	551	0.86	6.1	9.5	0.10	0.06	1121	103.7	10231	86785	3.5	1.8
	8.82		1245	664	0.86	5.9	9.3	0.10	0.06	1588	245.7	11159	162481	3.7	1.8

N = N value of standard penetration test, Wu = weight of super structure, H = distance between mass m_1 and rocking center (see Fig.1), T_{10} = natural period of pier with rigid base, μ_u = ductility capacity, γ = pastic stiffness ratio of Q-hyst model, Q_{y1},Q_{y2},Q_{y3}=yield strength of pier,sway and rocking of pile foundation

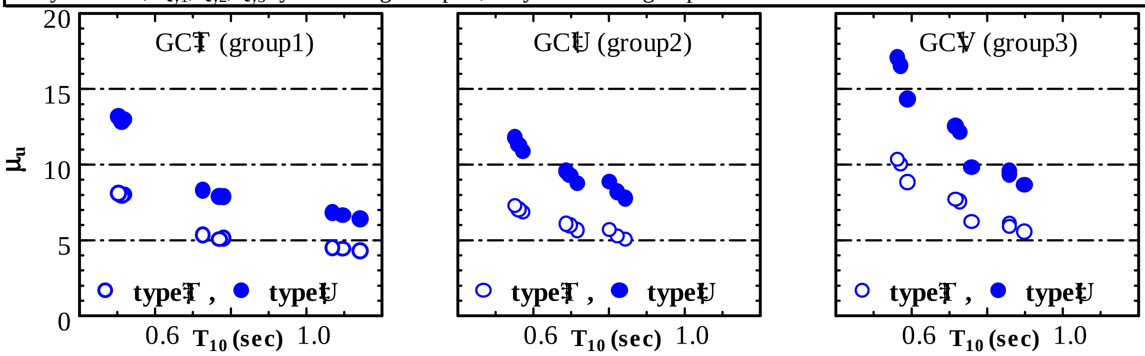


Fig.4: Relationship between ductility capacity and natural peried T_{10} for RC pier

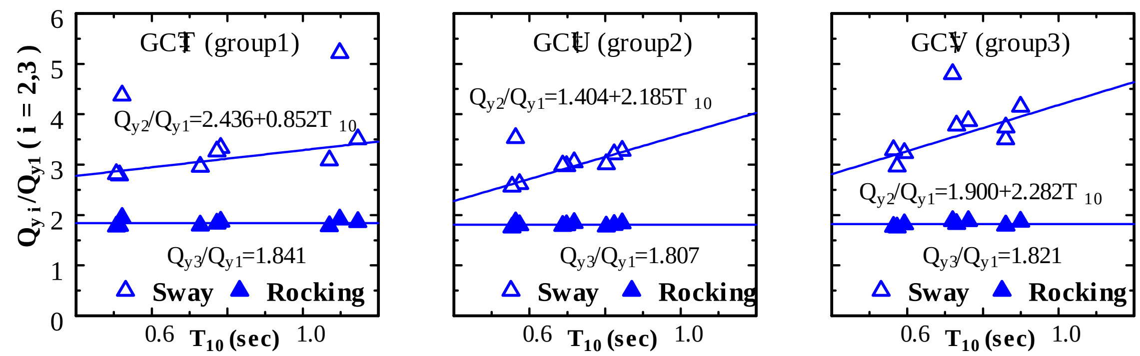


Fig.5: Ratio of yield strength of Q_{y2} and Q_{y3}(=$Q_{y\theta}$/H) for foundation to Q_{y1} for pier;Q_{y2}/Q_{y1} and Q_{y3}/Q_{y1}

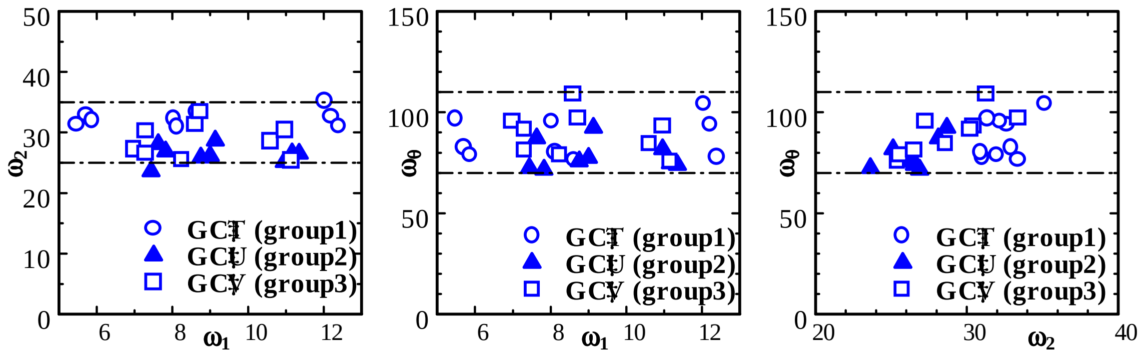


Fig.6: Relationship among natunal circular freguencies ω_1 for pier, ω_2 and ω_θ for foundation

3. INPUT EARTHQUAKE AND EQUATION OF MOTION

3.1 Input Earthquake

In this study twenty-one artificial earthquakes, the same as our previous study [Hirao et al 1997], are used as the input earthquake motions of inelastic response analyses for the S-R SDOF system and R-B SDOF system. Here, it is noted that the acceleration response spectrum of each artificial earthquake coincides with the spectrum of the type Ⅰ (far site) and type Ⅱ (near site) earthquakes of level 2 for the three soil conditions (GC Ⅰ, Ⅱ and Ⅲ) in the Japanese code, respectively. The major data on these simulated earthquakes are summarized in Table 2.

3.2 Equation of Motion

The equation of motion for the above-mentioned S-R 3DOF system, subjected to an earthquake motion, can be written as follows:

$$\boldsymbol{M}\ddot{x} + \boldsymbol{C}\dot{x} + \boldsymbol{Q}(\,x\,) = -\boldsymbol{m}\ddot{x}_0 \qquad (1)$$

where $\ddot{x}_0$ is acceleration of an input earthquake motion; the dots represent differentiation with time t; and the matrices **M**, **C**, **Q**, **m** and **x** are as follows:

Table 2: Data of artificial earthquakes

Earthquake			Number	GC	M	D (km)	A_{max} (gal)	V_{max} (kine)	P_t ($gal^2 sec$)
	Type	Group							
Level 2	type‡T	group1	1	‡ T	8.0	100	304.58	69.44	196447
			2	‡ T	8.0	200	338.56	75.56	216758
			3	‡ T	8.0	300	342.91	71.83	234428
		group2	4	‡ U	8.0	100	370.49	98.95	314440
			5	‡ U	8.0	200	398.13	100.44	341330
			6	‡ U	8.0	300	413.68	96.40	359053
		group3	7	‡ V	8.0	100	428.71	136.11	471454
			8	‡ V	8.0	200	449.27	144.09	508142
			9	‡ V	8.0	300	482.53	137.65	527685
	type‡U	group1	10	‡ T	7.2	5	707.29	82.87	415208
			11	‡ T	7.2	10	617.57	74.96	534441
			12	‡ T	7.2	20	756.99	75.42	540967
			13	‡ T	7.2	30	775.68	80.79	552861
		group2	14	‡ U	7.2	5	597.73	122.65	486605
			15	‡ U	7.2	10	618.76	124.30	575874
			16	‡ U	7.2	20	722.91	136.39	588940
			17	‡ U	7.2	30	699.58	130.99	653443
		group3	18	‡ V	7.2	5	535.42	144.58	451736
			19	‡ V	7.2	10	502.87	141.39	482750
			20	‡ V	7.2	20	607.90	150.49	496682
			21	‡ V	7.2	30	584.24	146.19	528252

Level 2 = earthquake for checking lateral seismic strength, type‡ Tand type‡ U far sit and near site earthquake, GC = soil condition (ground type), M = magunitude, D = epicentral distance, A_{max} and V_{max} = maximum acceleration and velocity, P_t = total power of acceralation wave

$$
\boldsymbol{M}=\begin{bmatrix} m_1 & m_1 & m_1 \\ m_1 & m_1+m_2 & m_1 \\ m_1 & m_1 & m_1+m_3 \end{bmatrix},\ \boldsymbol{C}=\begin{bmatrix} c_1 & 0 & 0 \\ 0 & c_2 & 0 \\ 0 & 0 & c_3 \end{bmatrix},\ \boldsymbol{Q}=\begin{bmatrix} Q_1(x_1) \\ Q_2(x_2) \\ Q_3(x_3) \end{bmatrix},\ \boldsymbol{m}=\begin{bmatrix} m_1 \\ m_1+m_2 \\ m_1 \end{bmatrix},\ \boldsymbol{x}=\begin{bmatrix} x_1 \\ x_2 \\ x_3 \end{bmatrix} \qquad (2)
$$

where $\ddot{x}_0$ is the acceleration of input earthquake motion; the dots of x represent differentiation with respect to time t; and m_3, c_3 and $Q_3(x_3)$ are as follows:

$$
m_3 = I_\theta / H^2, @\ c_3 = c_\theta / H^2, @\ Q_3(x_3) = Q_\theta(\theta)/H \qquad (3)
$$

Moreover integrating Eq. (1) multiplied by $\dot{\mathbf{x}}^T dt$ from the left side, an energy equilibrium equation is obtained by:

$$
\int \dot{x}^T \boldsymbol{M}\ddot{x}dt + \int \dot{x}^T \boldsymbol{C}\dot{x}dt + \int \dot{x}^T \boldsymbol{Q}(x)dt = -\int \dot{x}^T \boldsymbol{m}\ddot{x}_0 dt \qquad (4)
$$

where superscript ^{T} represents the transportation of a matrix.

4. DAMAGE INDEX, REDUCTION FACTOR AND REQUIRED SEISMIC ITENSITY

3.1 DAMAGE INDEX

In this study, modified Park-Ang's damage index D in Eq. (5) is employed as a standard to evaluate damage of a RC pier, and mean value of the coefficient β(=0.15) [Park et al. 1985] is adopted.

$$
D = \{(\mu_d - 1) + \beta \cdot \mu_h\}/(\mu_u - 1), \quad \mu_h = E_h/(Q_{1y} \cdot x_{1y}) \qquad (5)
$$

in which μ_d, μ_u and μ_h are the displacement ductility, ductility capacity and energy ductility; Q_{1y} and x_{1y} are the yield strength and displacement; and E_h is the cumulative hysteretic energy of the pier.

3.2 REDUCTION FACTOR

In this study, required yield strength ratio q_{yr} is defined as the required value of yield strength ratio q_y, by which the value of damage index D in Eq.5, for a RC pier, will result in a designated value D_r [Hirao et al. 1995, 1997]. Then the required yield strength ratios q_{yrI} for S-R 3DOF system in Fig.2 and q_{yrF} for R-B SDOF (rigid based single degree of freedom) system are obtained from the repetition of ordinary inelastic energy response analyses of Eqs. (1) and (4). After that reduction factor R for the pier is obtained as follows [Krawinkler et al. 1992]:

$$
R = 1/q_{yr} = Q_{1ye}/Q_{1yr} \qquad (6)
$$

where Q_{1yr} is the required yield strength for the RC pier; Q_{1ye} is the yield strength required of the pier, in order to respond elastically to an earthquake motion.

3.3 REQUIRED SEISMIC INTENSITY

After calculating the yield strength Q_{1yr} by Eq. (7) [Hirao et al.1995], the required seismic intensity (coefficient) k_r of the pier, provided that the value of modified Park-Ang's damage index D of the pier is equal to a designated one D_r (=0.4, 0.7 and 1.0), is obtained from Eq. (8). In this study the required ultimate seismic intensity k_{ur} is also defined as in Eq.(9), in order to discuss the design seismic intensity of RC piers (see **5.2**).

$$
Q_{1yr} = Q_{1ye}/R = m_1 \cdot \bar{s}_a \cdot \ddot{x}_{0\,max}/R \qquad (7)
$$

$$
k_r = Q_{1yr}/(m_1 \cdot g) = Q_{1yr}/W_1 \qquad (8)
$$

$$
k_{1urI} = Q_{1ur}/W_1, \quad Q_{1ur} = \{1 + (\mu_{1d} - 1)\}Q_{1yr} \qquad (9)
$$

in which $\bar{s}_a$ and $\ddot{x}_{0max}$ are the pseudo acceleration response factor of the pier and maximum acceleration of an input earthquake motion; and g and W_1 are the acceleration of gravity and weight of mass m_1. And Q_{1ur} and μ_{1d} are maximum restoring force and displacement ductility of the pier, corresponding to the required yield strength Q_{1yr}.

5. NUMERICAL RESULTS

The values of N=30, 20 and 5, employed as the soil conditions (ground types) for seismic design of RC piers with pile foundation, correspond to the ones of soil conditions GC Ⅰ (ground type Ⅰ: hard soil), GC Ⅱ (ground type Ⅱ: median soil) and GC Ⅲ (ground type Ⅲ: soft soil) in the Japanese code. In this study, therefore, each seven earthquakes (three of the type Ⅰ and four of the type Ⅱ) of the group 1 for GC Ⅰ (hard soil), group 2 for GC Ⅱ (median soil) and group 3 for GC Ⅲ (soft soil) in Table 2 are used as the input earthquake motions for the response analyses of each nine RC piers with soil condition N=30, N=20 and N=5 in Table 1, respectively. Then both of the required seismic intensity (coefficient) k_{rI} for the S-R (3DOF) system and k_{rF} for the R-B (SDOF) system and other responses of each pier, corresponding to the designated D_r values (D_r=0.4, 0.7 and 1.0), are obtained from the inelastic energy response analyses. Here the coefficient of variation of k_r value of each pier, among the three of type Ⅰ and four of type Ⅱ earthquakes is smaller than 0.2 for almost all the piers. Therefore, the mean value of the k_r for each earthquake group will be shown, hereafter.

5.1 Effect of Foundation Interaction on k_r

In order to discuss the effect of foundation-structure interaction on the required seismic intensity K_r of the RC piers, Fig.7 illustrates the ratio of the k_{rI} of S-R 3DOF system to the k_{rF} of R-B SDOF system, i.e., k_{rI}/k_{rF} (= R_F/R_I =Q_{1yrI} /Q_{1yrF}: see Eqs. (7), (8)), against the natural period T_{10} of each pier. In the figure, the difference of k_{rI}/k_{rF} value in the soil conditions (N values or earthquake groups), earthquake types (type Ⅰ, type Ⅱ) and designated D_r values (D_r=0.4, 0.7, 1.0) are compared. Here it is noted that, when the value of k_{rI}/k_{rF} for a pier is larger than 1.0, the foundation interaction affects disadvantageously on its damage/or seismic force, and vice versa. From Fig.7, it can be seen that the ratio k_{rI}/k_{rF} shows larger or smaller value than 1.0, relating to the earthquake type, soil condition GC and D_r value. It is also found that the values of k_{rI}/k_{rF} for almost all the piers on the soft soil (GC Ⅲ: N=5) become larger than 1.0, not relating to earthquake types and designated D_r values. On the contrary, the ratios k_{rI}/k_{rF} for almost all the piers on the hard soil (GC Ⅰ: N=30) show smaller value than 1.0. However, as for the piers on the median soil (GC Ⅱ: N=20), the

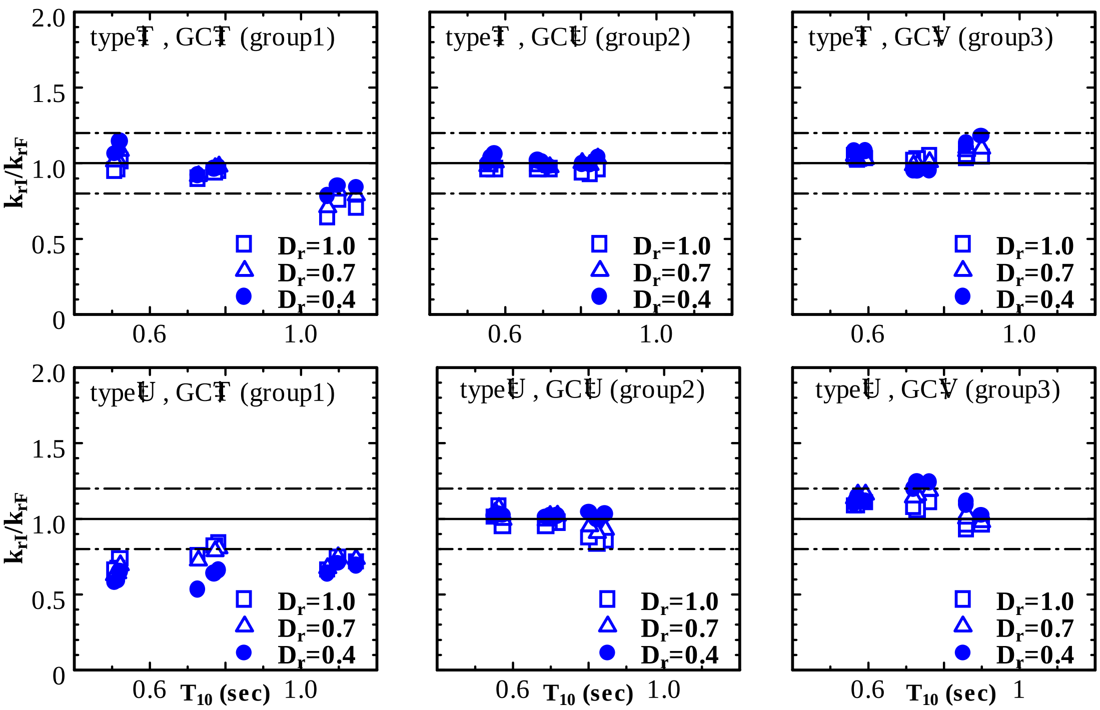


Fig.7: Ratio of k_{rI}/k_{rF} of RC piers, comparing the difference in earthquake types(type Ⅰ, type Ⅱ) soil conditions(GC= Ⅰ, Ⅱ, Ⅲ) and designated D_r values(Dr=0.4, 0.7, 1.0)

k_{rI}/k_{rF} values in case of the D_r=0.4 are relatively close to 1.0, while the values in case of the Dr=1.0 becomes rather smaller than 1.0. Therefore, it is known that the effect of foundation-structure interaction on the required seismic intensity /or damage of the RC pier depends mainly on the type of soil condition (ground type) and somewhat on the type of earthquake and D_r value.

5.2 Comparison of code seismic intensity with required one

On the seismic design concept for type B (important) bridges that the damage of a bridge subjected to level 2 earthquakes should be limited within a repairable state, the bridge pier in Japan is designed so that the following requirement is satisfied:

$$P_a \geq k_{he} \cdot W \qquad (10)$$

where P_a is lateral capacity (ultimate strength) of the pier; k_{he} is equivalent seismic intensity (coefficient) reduced by the energy-equal assumption with allowable displacement ductility μ_a; and W is the equivalent weight.
In this place, therefore, the seismic intensity k_{he}, used for the design of RC piers described before, is compared with the required ultimate seismic intensity k_{urI}, as defined in Eq.(9) , for the designated damage Dr=0.4, being close to the repairable limit [Ghobarah et al. 1998] and D_r =1.0 (collapse limit). It is noted that, if the k_{he} of a pier is larger than the k_{urI} for D_r=0.4, the pier is designed safely for the criterion that the value of modified Park-Ang's damage index D of the pier is equal to or smaller than 0.4 (repairable limit), and vice versa.
Fig.8 shows the ratio of k_{urI}, for the S-R 3DOF system taking account of the foundation interaction, to k_{he}, i.e., k_{urI}/k_{he}, comparing the difference of ratio k_{urI}/k_{he} in the D_r values (D_r=0.4, 1.0), soil conditions (GC= Ⅰ, Ⅱ, Ⅲ) and earthquake types (type Ⅰ, type Ⅱ). It can be seen from Fig.8 that, in the case of D_r=0.4, the ratio k_{urI}/k_{he} of all the piers on the soft soil (GCⅢ: N=5) and median soil (GCⅡ: N=20) indicates larger value than 1.0, for both the type Ⅰ and type Ⅱ earthquakes. Also the k_{urI}/k_{he} value for some of the piers on the hard soil (GCⅠ: N=30) subjected to type Ⅰ earthquake shows larger value than 1.0. Contrary to this, in the case of D_r=1.0, the value of k_{urI}/k_{he} for almost all the piers on the hard soil and median soil is smaller than 1.0, and even for the piers on the soft soil , some of the k_{urI}/k_{he} values become smaller than 1.0. From the figure, it is also found that the k_{urI}/k_{he} value depends on the soil condition and natural period T_{10} of the pier. That is, the value of k_{urI}/k_{he} gets larger as the soil becomes softer and with decreasing value of the T_{10}. And the k_{urI}/k_{he} value depends on the type of earthquake, i.e., the ratio k_{urI}/k_{he} for type Ⅱ earthquake shows larger value

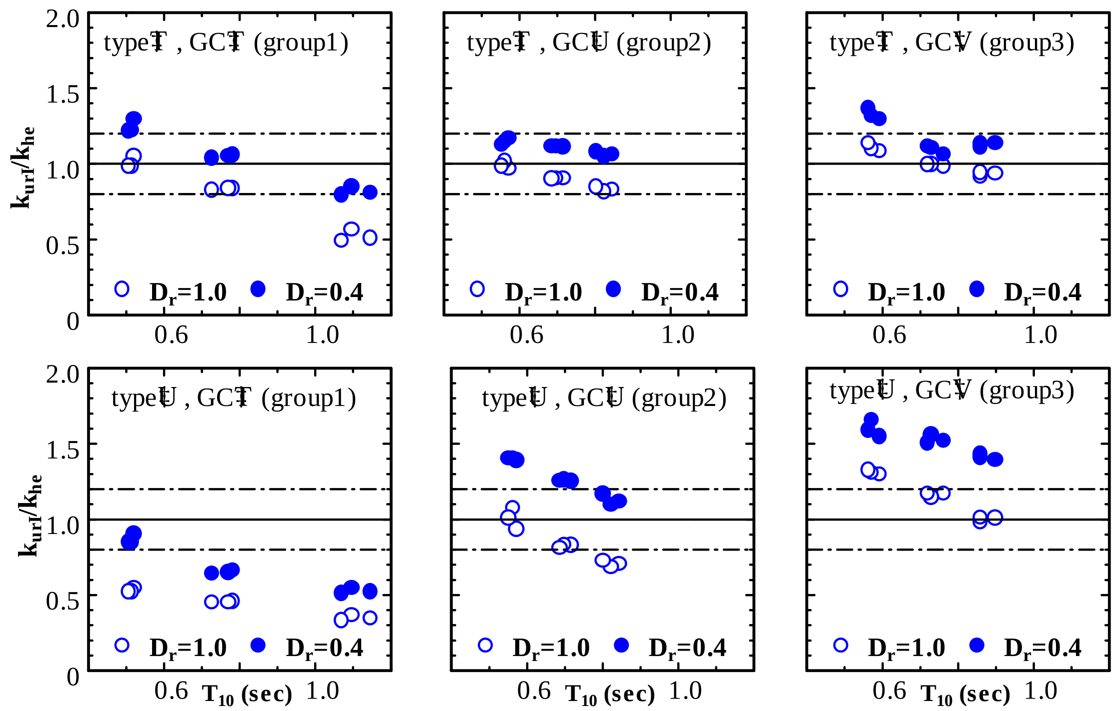


Fig.8: Ratio of k_{urI}/k_{he} for pier, comparing the difference in earthquake types, soil conditions and D_r values

than the one for type Ⅰ earthquake. As a result, it is said that the equivalent seismic intensity k_{he} for a single column RC pier, is not enough large to limit the value of modified Park-Ang's damage index D within the range of D=0.4 (repairable limit), especially for the piers on the soft soil (GCⅢ) subjected to an type Ⅱ earthquake.

6. CONCLUSIONS

In this study, required seismic intensity k_{rI} for the S-R system and k_{rF} for R-B SDOF system are obtained from inelastic energy response analyses, provided that the value of modified Park-Ang's damage index D of a pier is equal to a designated one D_r (=0.4, 0.7, 1.0). Then, illustrating the ratio k_{rI}/k_{rF}, the effect of foundation interaction on the required seismic intensity (force) of RC piers is examined. Equivalent seismic intensity k_{he} is also compared with the required ultimate seismic intensity k_{urI} in case of the designated damage Dr=0.4, being close to the repairable limit, and D_r =1.0 (collapse limit), in order to discuss the suitableness of seismic design force of RC piers in the Japanese code.

The main results obtained in this study are summarized as follows:

(1) The effect of foundation-structure interaction on the required seismic intensity of a RC pier with pile foundation depends mainly on the type of soil. The effect also depends a little on the type of earthquake and designated value of modified Park-Ang's damage index D (D_r value). And the values of k_{rI}/k_{rF} for almost all the piers on the soft soil (GCⅢ: N=5) become larger than 1.0. On the contrary, the ratios k_{rI}/k_{rF} for almost all the piers on the hard soil (GCⅠ: N=30) show smaller value than 1.0. However, as for the piers on the median soil (GCⅡ: N=20), the k_{rI}/k_{rF} values are relatively close to 1.0.

(2) The equivalent seismic coefficient k_{he}, for a single column RC pier with pile foundation, is not enough large to limit the value of modified Park-Ang's damage index D within the range of D=0.4 (repairable limit), especially for the pier on the site of soft soil (GCⅢ) and subjected to typeⅡ earthquakes.

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