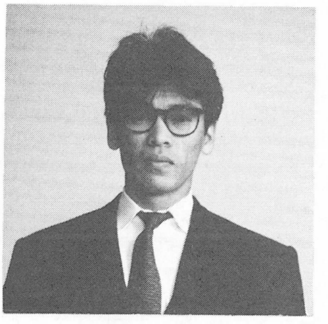
CONCRETE LIBRARY OF JSCE NO. 10. DECEMBER 1987

INELASTIC RESPONSE OF REINFORCED CONCRETE FRAME STRUCTURES SUBJECTED TO EARTHQUAKE MOTION





Atsuhiko MACHIDA **Hiroshi MUTSUYOSHI** **Kazuhisa TSURUTA**

ABSTRACT

In order to clarify inelastic behaviors of reinforced concrete frame structures subjected to earthquake motion, shaking table tests and pseudodynamic tests were carried out using small scale two-story one-bay reinforced concrete bridge piers, and the inelastic response analyses based on one component model were conducted. The response behaviors of reinforced concrete frame structures depend strongly on the ultimate failure mode of each member though the whole structure may not collapse. Therefore, the restoring force-displacement model, which can represent well the strength and the displacement in the ultimate state, that is ductility, of each member was proposed to obtain accurately the inelastic responses of reinforced concrete frame structures. Using the proposed restoring force model, the response behaviors of reinforced concrete frame structures in an inelastic range could be calculated accurately even if shear failure occurred in some member. From the test and calculated results, to design reinforced concrete rigid-frame piers as a seismic resistant structure, beams as well as columns need to have adequate ductility.

A. Machida is a professor of Department of Construction Engineering at Saitama University, Urawa, Japan. He was awarded the JSCE prize (Yoshida Prize) in 1979 and 1987. He received his doctor of Engineering Degree in 1976 from the University of Tokyo. His research interests include test methods for properties of concrete and seismic design of reinforced concrete structures.

H. Mutsuyoshi is an associate professor of Department of Construction Engineering at Saitama University, Urawa, Japan. He received his Doctor of Engineering Degree in 1984 from the University of Tokyo. He was awarded the JSCE prize (Yoshida Prize) in 1987. His research interests include seismic design of reinforced concrete structures and dynamic response of reinforced concrete members.

K. Tsuruta is an engineer of Metropolitan Expressway Public Co.. He received his master of Engineering Degree in 1986 from Saitama University. He was awarded the JSCE prize (Yoshida Prize) in 1987.

1. INTRODUCTION

Recently, many reinforced concrete rigid-frame highway and railway bridge piers have been constructed in Japan and also damaged due to earthquakes. For example, in Miyagi-ken-oki Earthquake of 1978, it was reported that the columns or the beams of the reinforced concrete rigid-frame piers of the elevated railway bridges were remarkably damaged due to the earthquake load greater than the design earthquake load[2,3]. Generally, in the case of a statically indeterminate structure such as reinforced concrete rigid-frame piers, the whole structure will not collapse even if one of the members, which constitute a frame structure, fails perfectly. However, it is considered that the inelastic response behaviors of the whole structure are influenced by failure of the member. It is very important to make clear the seismic properties in the plastic range of reinforced concrete structures because a design load due to earthquakes and a safety for earthquake depends strongly on the mechanical properties in the plastic range. Therefore, in order to design reasonably reinforced concrete rigid-frame piers as an earthquake-proof structure, it is necessary to examine in detail the behaviors of structures in the plastic range. Since most of reinforced concrete structures have been generally designed by the static force method using the seismic coefficient, the safety for earthquake when subjected to severe earthquakes greater than the design earthquake load, the effects of strength and ductility of each member on the response behaviors and the failure mechanism of the structure have not yet been satisfactorily clarified.

On the other hand, many studies on reinforced concrete frame structures have been conducted in the field of building structures because most of the structures are frame structures. Since sectional characteristics, amount of reinforcements and axial force of reinforced concrete rigid-frame piers are very different from those of building structures, however, it is necessary to study reinforced concrete rigid-frame piers independently of building structures.

The objective of this paper is to clarify experimentally and analytically the inelastic response behaviors of reinforced concrete rigid-frame piers subjected to strong ground motion. In order to investigate the inelastic behavior of reinforced concrete frame structures subjected to earthquake motion, simulated earthquake tests using a shaking table and pseudodynamic tests were carried out using small scale two-story one-bay reinforced concrete bridge piers, which are widely used for the elevated highway and railways in Japan, and inelastic response analyses based on one component model were conducted.

2. OUTLINE OF EXPERIMENT

The test structures are two-story one-bay reinforced concrete frames which are intended to represent a portion of typical bridge piers used in the elevated railways of the Tohoku-Shinkansen. **Figure 1** shows the dimensions of the test structures, and **Table 1** describes the details of them. Deformed bars of 6 mm were used for the main reinforcements in the columns and the second-level beams, and those of 3 mm were used for the web reinforcements. The main and web reinforcement ratios were determined on basis of the Standard Design for Structures of Japan National Railways[4]. The first-level beam was designed assuming that each member of the test structure would fail showing the following failure modes; 1)flexural failure will occur at the bottom of the first-level column (structures RD-1 and RP-1), 2)flexural failure will occur in the first-level beam (RD-3), 3)shear failure will occur after yielding of main reinforcements in the first-level beam (RD-4 and RP-4). To produce the above failure modes, the tensile reinforcement ratio and the web reinforcement ratio in the first-level beam were changed as shown in **Table 1.** The maximum size of the coarse aggregate of concrete was 5 mm for the columns and the beams, and 12.5 mm for the footing.

In every test, a weight of 963 kgf was installed at the top of each second-level column in such a way that this weight could be free to rotate around its central axis to eliminate the inertia force due to rotation. The axial stress of each column caused by the weight was 9.6 kgf/cm^2. A general view of the test set-up is shown in **Fig.2.** Three structures, RD-1, RD-3 and RD-4, were tested under simulated earthquakes and two structures, RP-1 and RP-4, were tested pseudodynamically.

In the simulated earthquake tests, the first 10 seconds of EL CENTRO-NS 1940 earthquake was repeated three times continuously. To excite the test structures into an inelastic range(about three or four times of yield displacement), the original time scale was compressed by a factor of 2 while the maximum base acceleration was amplified to 0.8g. For each structure, the free vibration test was conducted to measure the natural frequency and the initial damping factor, and then the simulated earthquake test was carried out. The absolute acceleration at each story was measured by accelerometers. The base acceleration was measured by the accelerometer mounted on the base. All these accelerometers were oriented parallel to the base motion. Each story displacement with respect to the base was measured by linear voltage differential transducers, and the strains of the reinforcing bars at the root of the column and of the beam were measured by wire strain gages. All the data obtained during the experiments were recorded on a data recorder, and then were transformed into digital data through the A/D converter. The time interval of sampling for transforming analog data into digital data was 0.0005 sec.

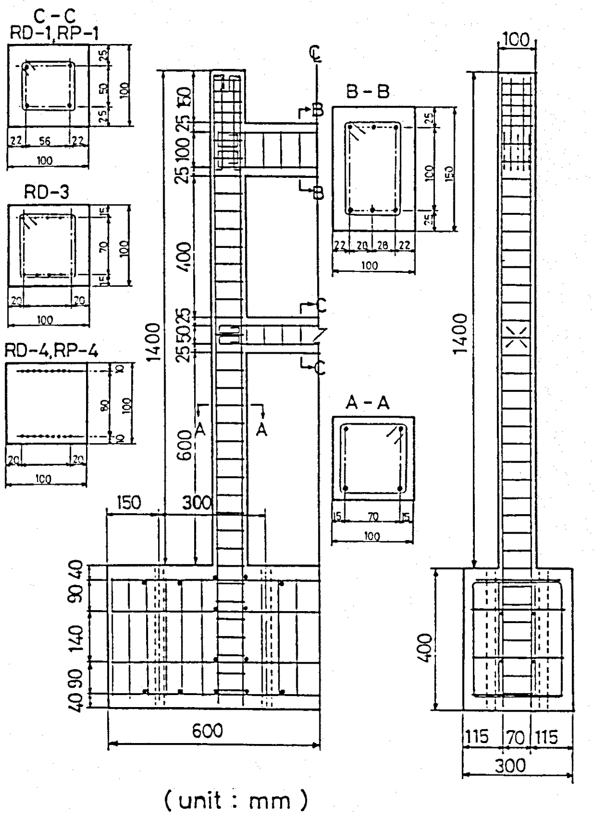


Fig.1 Test structure

The pseudodynamic test proceeds in a stepwise manner under a step by step integration procedure. In each step, the computed displacements are quasi-statically imposed on the test structure by means of a computer controlled electrohydraulic actuator. The restoring forces, measured at the end of a step, are then used to compute the displacement response in the next step based on the analytically prescribed values of the mass of the system and the initial damping factor as well as on a numerically specified base acceleration. In this case, the base accelerations obtained from the simulated earthquake tests were used to compare the response behaviors of structures under dynamic earthquake loading with those under static pseudodynamic loading. This process is repeated until the entire response history is obtained. Since all the weight(1956 kgf) of the mass, installed at the top of each second-level column, was much larger than that of the structure(95 kgf), it was concluded that the first mode is dominant for the test structures. Therefore, the whole system was assumed as a single degree-of-freedom. The pseudodynamic method is shown in **Fig.3.**

3. INFLUENCE OF STRENGTH AND DUCTILITY OF EACH MEMBER ON RESPONSE BEHAVIORS OF STRUCTURES

From the test results, structures RD-1, RD-3 and RP-1 failed showing a typical flexural failure at the both ends of the first-level beam and at the bottoms of the first-level columns, while structures RD-4 and RP-4 failed showing finally a diagonal tension failure after yielding of reinforcing bars in the first-level beam. It is considered that the response behaviors of the structures must depend on the failure mode of the members because the failure mode varied remarkably with the test structures. In order to make clear the influence of the failure mode of the members on the response behaviors, base-shear and top displacement curves were examined. The base shear was obtained from the sum of the product of the mass and the measured acceleration at each level.

Figure 4 shows the relations between the base shear and the top displacement obtained from the simulated earthquake tests. The base shear-displacement curves of RD-1, in which flexural failure occurred in the first-level beam, show large energy dissipation, while those of RD-4, in which shear failure occurred finally in the first-level beam, show that the capacity of energy dissipation tends to decrease with time. That is, the failure mode of the member influences significantly on the capacity of energy dissipation of the structure. The same result was obtained from the pseudodynamic tests(structures RP-1 and RP-4). These results indicate that the response behaviors of the structures depend strongly on the failure mode of each member.

Figure 5 shows the time histories of the second-level displacement. The point of "a", indicated in **FIg.5,** means the time when the structure became a statically determinate stage by forming six yield hinges in each member, and the point of "b" indicates the time when shear failure occurred in the first-level beam and then the load carrying capacity of the structure begins to decrease. Comparing the time histories of RD-1 with those of RD-3, in which flexural failure occurred in all members and shear failure didn't occur, the period of excitation and the displacement amplitude of both structures were approximately same after the point of "a" although the strengths of the first-level beams of both structures were

Table 1 Details of test structures

Common Members

Member Name	Tensile Reinforcement Ratio (%)	Web Reinforcement Ratio (%)	Relative Stiffness Ratio (*)
First-Level Column	0.75(D6X2)	0.29(D3)	1.00
Second-Level Column			1.24
Second-Level Beam	0.76(D6X2)		4.35

First-Level Beam

Specimen Name	Tensile Reinforcement Ratio (%)	Web Reinforcement Ratio (%)	Relative Stiffness Ratio (*)
RD-1	0.85(D6X2)	0.29(D3)	1.24
RP-1			
RD-3	0.43(D3X5)	0.058(D2)	1.21
RD-4	0.73(D3X9)	0.0	1.26
RP-4			

Note (*): The stiffness of the first-level column is the standard value (1.0).

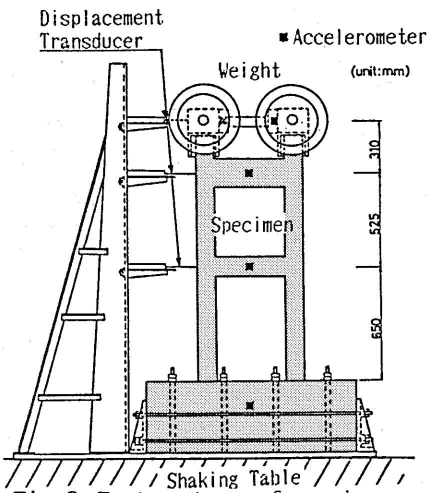


Fig.2 Test set up for simulated earthquake test

different. On the other hand, it was apparent from the time history of RD-4 that the period became longer and the displacement amplitude became larger after the point of "b" as compared with RD-1 and RD-3. That is, the response behaviors of the structure RD-4 changed remarkably because shear failure occurred in the first-level beam. From these results, it can be concluded that the beam as well as the column needs to have sufficient ductility without degradation of strength in order to design two-story reinforced concrete rigid-frame piers as a seismic resistant structure.

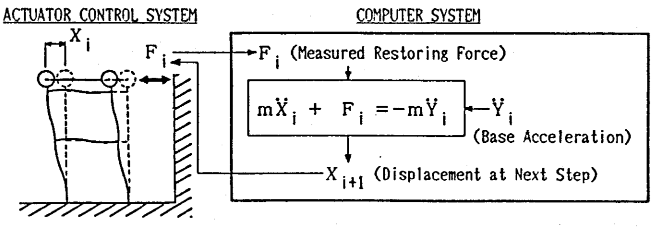


Fig.3 System for pseudodynamic test

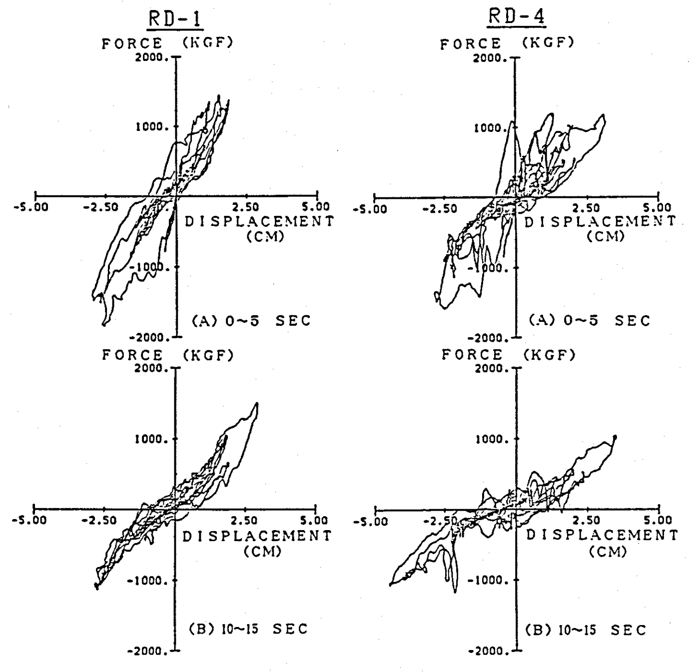


Fig.4 Base shear-displacement relationship obtained from simulated earthquake test(RD-1 and RD-4)

4.COMPARISON OF SIMULATED EARTHQUAKE TEST WITH PSEUDODYNAMIC TEST

To clarify whether the response behaviors of reinforced concrete frame structures under dynamic loading are equal to those under static loading, the base shear-displacement curves obtained from the simulated earthquake test (structure RD-1) and the pseudodynamic test(structure RP-1) were examined as shown in **Fig.6.** The displacement velocity under the simulated earthquake test is approximately 1000 times as much as that under the pseudodynamic test. The difference of such velocity is large enough to investigate the effect of loading rate. Moreover, it was confirmed from the fourier spectrum of base shear and response displacement that only the first mode was dominant and the effect of high order mode was hardly included in the base shear-displacement relations. The broken line shown in **Fig.6** indicates the base shear calculated statically by the virtual work method. The maximum base shear obtained from the pseudodynamic test agreed with the statically calculated value, while the maximum base shear obtained from the simulated earthquake test was about 20 % higher than the statically calculated one. In order to clarify this cause, the time histories of the base shear, the displacement and the strains of the reinforcing bars of each member were investigated. It was observed that a high magnitude of strain rate was produced at the moment when the reinforcing bars of each member reached the yield strain. In the case of RD-1, for example, the strain rate at yielding of the reinforcing bar was from 10 to 20 %/sec. It is well known that the yield stress of a normal reinforcing bar increases by about 20 % when such strain rate is produced. From above facts, it was considered that the rise of the base shear could be

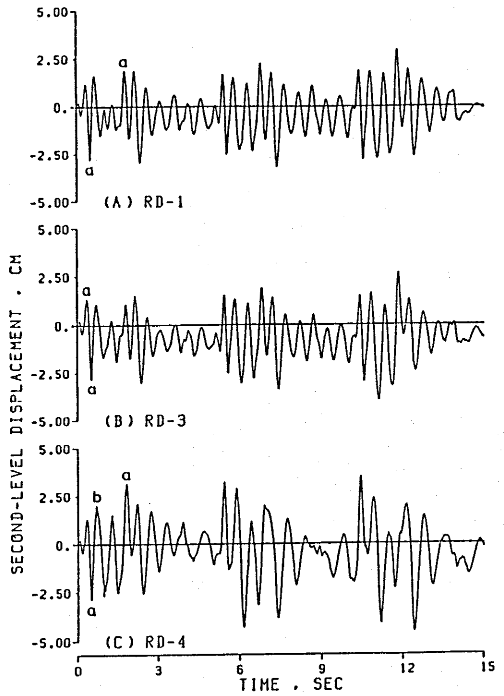


Fig.5 Time history of displacement measured from simulated earthquake test(RD-1, RD-3 and RD-4)

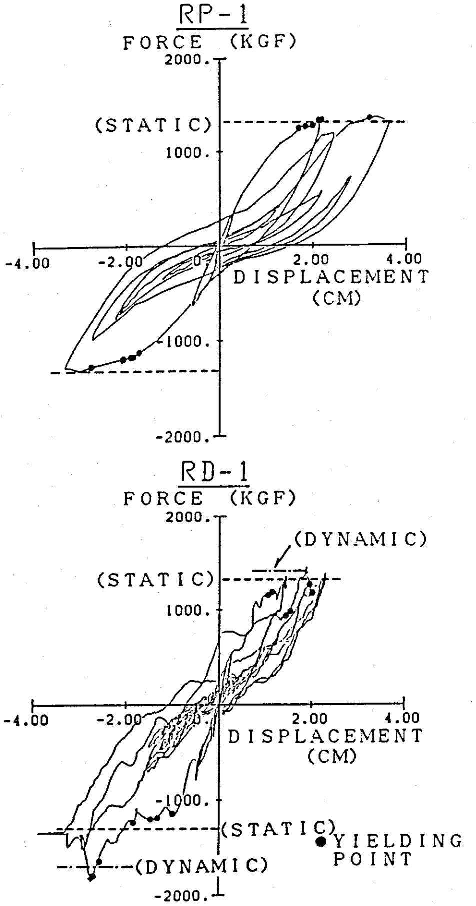


Fig.6 Influence of loading rate on base shear

attributed to the increase of the strength of each member due to strain rate effect[5]. Therefore, the maximum base shear was calculated again using the real yield stresses, which correspond to the measured strain rate of the reinforcing bars of each member. The calculated results are shown in **Fig.6.** The base shear calculated dynamically agreed well with the experimental values. That is, when subjected to dynamic loading, the strength of a reinforced concrete frame structure as well as a single reinforced concrete column increases due to strain rate effect of reinforcing bars. Note that the response behaviors obtained from the pseudodynamic test may not represent the real ones of reinforced concrete structures subjected to strong ground motion. In order to calculate accurately a response acceleration and a base shear of a reinforced concrete structure subjected to severe earthquakes, it is necessary to use the dynamic restoring force model based on strain rate effect[6].

5. NONLINEAR RESPONSE ANALYSES OF REINFORCED CONCRETE FRAMES BASED ON ONE COMPONENT MODEL

5.1 Analytical Model and Response Analysis

In order to obtain accurately the response behaviors of the whole structure and each member, the inelastic response analyses based on one component model[8] were carried out. The one component model, proposed by Giberson, consists of a linearly elastic member with two equivalent nonlinear springs at the member ends as shown in **Fig.7.** The rotational deformation of a member due to bending moment is expressed as the sum of the flexural deformation of the linear elastic member(EI:elastic flexural rigidity) and the rotational deformation of the two equivalent nonlinear springs(K_{pA}, K_{pB} :spring constant). The shear deformation of a member is represented by the elastic shear spring(Ks:spring constant) at the center of the member. The relation between the forces(M_A, M_B, Q_A, Q_B) and the deformations($\theta_A, \theta_B, U_A, U_B$) at the point of the joint of the member is indicated in **Eq.(1),**

$$\{M_A, M_B, Q_A, Q_B\}^T = [K]\ \{\theta_A, \theta_B, U_A, U_B\}^T \text{ --------(1)}$$

in which, [K]:a stiffness matrix of a member. The relation between the deformations of the flexural springs(τ_{pA}, τ_{pB}) and those of the point of the joint is shown in **Eq.(2).**

$$\{\tau_{pA}, \tau_{pB}\}^T = [C]\ \{\theta_A, \theta_B, U_A, U_B\}^T \text{ -----------(2)}$$

The matrices of [K] and [C] are as follows,

$$[K] = [C]^T\ [B]^T\ [F]^{-1}\ [B]\ [C]\ , \quad [F] = \frac{\ell'}{6EI}\begin{vmatrix} 2+p_A & -1+r \\ -1+r & 2+p_B \end{vmatrix}$$

in which, $p_A = \frac{6EI}{\ell'}\ \frac{1}{K_{pA}}$, $p_B = \frac{6EI}{\ell'}\ \frac{1}{K_{pB}}$, $r = \frac{6EI}{2\ell'}\ \frac{1}{Ks}$,

$$[B] = \begin{vmatrix} 1+a_A & a_A \\ a_B & 1+a_B \end{vmatrix}$$

$$[C] = \begin{vmatrix} 1 & 0 & 1/\ell & -1/\ell \\ 0 & 1 & 1/\ell & -1/\ell \end{vmatrix}$$

The inelastic moment-rotation relationship of a spring was calculated by means of ordinary flexural theory assuming the point of contraflexure at the center of the member. Furthermore, the rotation due to bond slip of the reinforcing bars from a beam-column joint was taken into consideration using Ohta's method[15]. This model was applied to all members of the test structure. As a restoring force model, the Takeda's model[11] was used for columns, and the Takeda's slip model[12], which could represent well the slip behaviors of the reinforcing bars from a beam-column joint and the hysteresis curve of reversed S type due to shear deformation, was used for beams. **Figure 8** shows the assumed restoring force models for columns and beams. In the Takeda's model, the stiffness during unloading(K_r) was defined as $(M_C + M_y)/(\theta_c + \theta_y)\ |\theta_y/\theta_m|^{\alpha}$ (see **Fig.8(a)**) where α was taken as 0.8 and 0.7 for columns and beams, respectively. In the Takeda's slip model, the stiffness during reloading was defined as $K_s = M_m/(\theta_m - X_o)\ (\theta_m/(\theta_m - X_o))^{\gamma}$ and $K_p = \eta M_m/\theta_m$ (see **Fig.8(b)**) where γ and η were taken as 2.0 and 1.0, respectively. The Newmark's method was used to integrate the equation of motion numerically, with a time interval of 0.0005 sec and β of 1/6. Damping matrix proposed by Wilson and Penzien was used[13]. In this case, the first mode damping factors measured by the free vibration tests were used until one of the members reached the yield rotation angle, and then the damping factors were taken as zero because only hysteretic damping was considered to be dominant after the yield displacement[5].

5.2 Problems In Applying One Component Model To Response Analyses

Figure 9(a) shows the measured and calculated time histories of the base shear and the top displacement for structure RD-3 of which the first-level beam failed finally in flexure. The calculated responses agreed generally with the measured ones except the maximum values of the base shear during the tests. The analytical model is very available for the frame structures if all members fail in flexure.

Table 2 shows the maximum and minimum values of base shear and displacement obtained from the simulated earthquake tests and analyses. The maximum and minimum values of the calculated base shear for RD-1, RD-3 and RD-4 were smaller than those of the measured one indicated in **Table 2.** The reason for this must be attributed to strain rate effect of reinforcing bars as described above.

Figure 10 shows the measured and calculated time histories for RD-4 in which shear failure occurred in the first-level beam. The period of the calculated responses is clearly shorter than that of the measured ones after shear failure occurred in the first-level beam. **Figure 11** shows the measured and calculated base shear-displacement curves. The measured base shear and the stiffness, defined by the slope of a line drawn through the points on the hysteretic curve corresponding to the maximum and minimum displacements, decreased remarkably as compared with the calculated ones after shear failure occurred in the first-level beam. That is, the restoring force model used in analysis can't simulate accurately the inelastic response behaviors of the structure in the case that shear failure was produced and then the load carrying capacity decreased suddenly in some member. This fact also indicates the limitation of the used restoring force model. Generally, in the case of a statically indeterminate structure such as reinforced concrete rigid-frame piers, the whole response behaviors of the structure depend strongly on the failure of the member even if the structure may not collapse. Therefore, the reliable restoring force model which can represent well the ultimate state of the member is required to clarify accurately the inelastic responses of reinforced concrete frame structures.

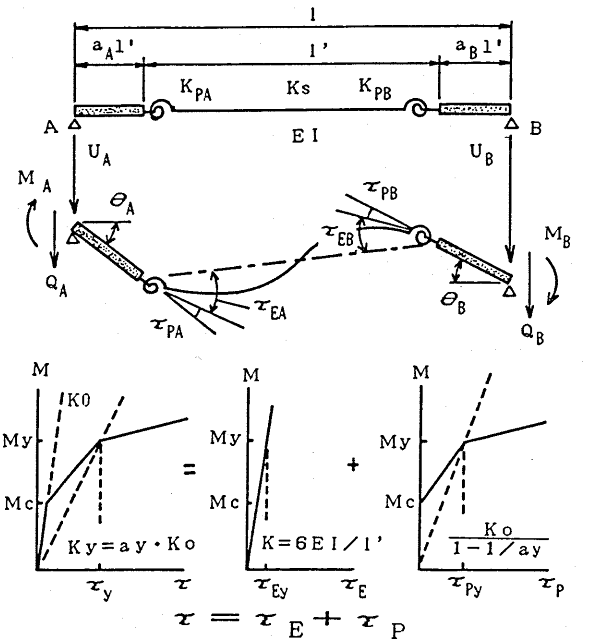


Fig.7 One component model

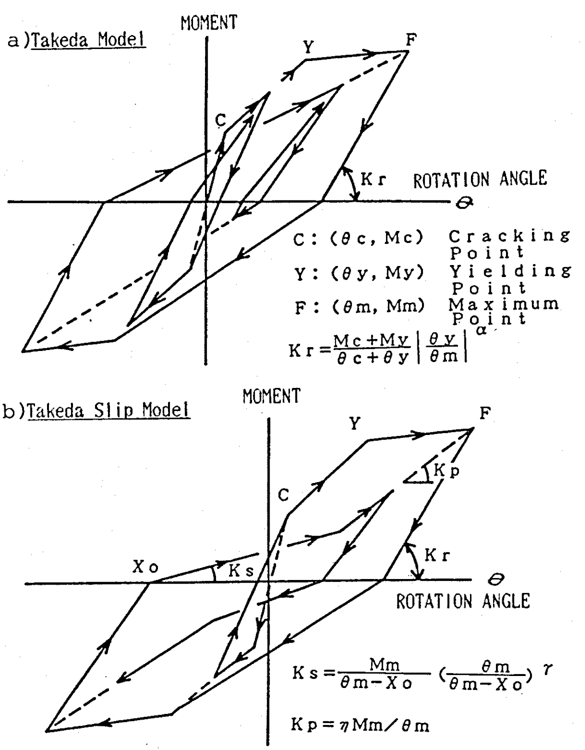


Fig.8 Assumed restoring force model for column(a) and beam(b)

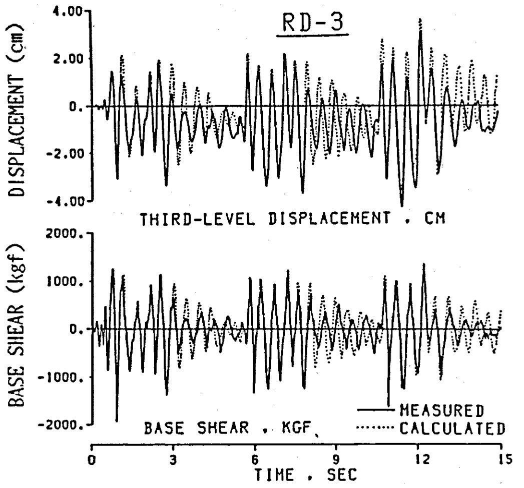


Fig.9 Measured and calculated displacement using one component model(RD-3)

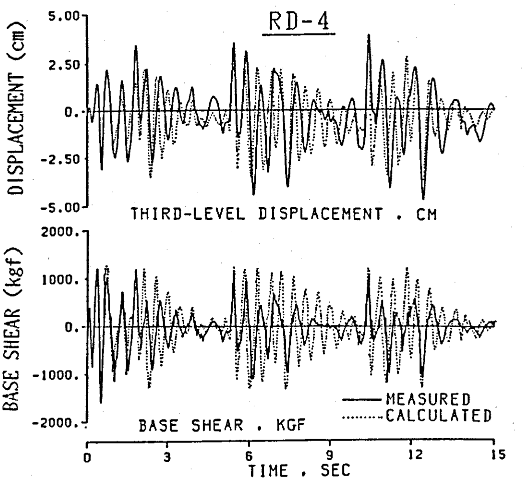


Fig.10 Measured and calculated displacement using one component model(RD-4)

Table 2 Maximum and minimum values obtained from tests and analyses

Specimen Name		Displacement (cm)				Base Shear (kg)	
		Second-Level		Third-Level			
		M‡	C‡‡	M	C	M	C
RD-1	Maximum	2.95	3.28	———	3.36	1549	1550
	Minimum	3.28	3.70	———	3.79	1898	1550
RD-3	Maximum	2.68	3.58	3.11	3.66	1405	1133
	Minimum	3.96	3.71	4.28	3.78	1975	1265
RD-4	Maximum	3.48	2.71	3.90	2.79	1215	1255
	Minimum	4.51	3.89	4.75	3.98	1595	1367

Note ‡)M : Measured ‡‡)C : Calculated

6.RESPONSE ANALYSES BASED ON ULTIMATE STATE OF MEMBER

6.1 Restoring Force Model Based on Ultimate State of Member

As described above, the response behaviors of the frame structures depend remarkably on the failure mode of each member, and they can be calculated satisfactorily by using the usual restoring force model in the case that all the members would fail in flexure. In the case that load carrying capacity of some member decreased suddenly due to the occurrence of shear failure, however, the response behaviors in inelastic range can't be calculated sufficiently by the usual restoring force model. In order to solve the above problem, the new restoring force model in consideration of the ultimate state of each member was proposed, and the response analyses using the proposed model were carried out.

Figure 12 indicates the new restoring force model. The new restoring force model was made by incorporating the restoring force characteristics of the ultimate state of a member in the usual restoring force model. The restoring force characteristics of the ultimate state, that is degrading of the load carrying capacity after the maximum strength, is represented by two lines. One is the line(segment U-D) showing the strength decreasing in negative slope after the

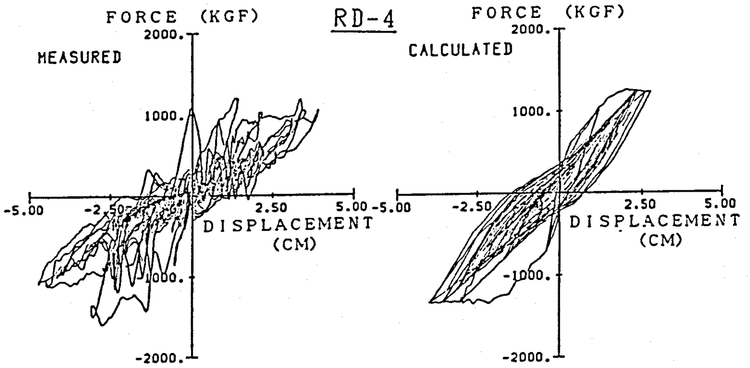


Fig.11 Measured and calculated base shear-displacement relationship

ultimate displacement, and another is the line(after point D) after the strength decreased(see **Fig.12**). In order to determine these two lines, it is necessary to define the ultimate rotational angle, that is ductility, of each member where the strength begins to decrease (point U). Ductility of reinforced concrete member has hardly been clarified satisfactory at the present stage. In this paper, the following equation[14], which can estimate accurately ductility factor, was adopted from the past studies.

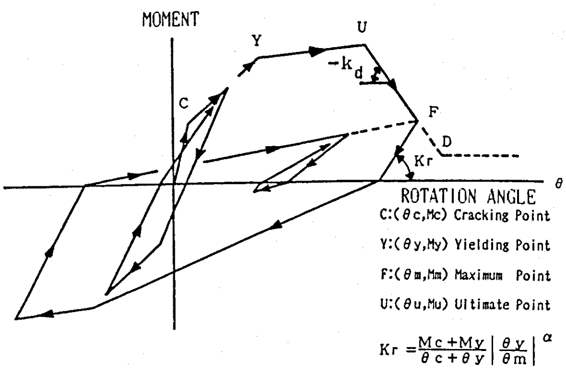


Fig.12 Proposed restoring force model considering ultimate state of member

$$\mu_u = \beta_0\ (1+\beta_t+\beta_w+\beta_N+\beta_a+\beta_n) \text{-----(3)}$$

where, μ_u :ductility factor(ultimate displacement/yield displacement)

$$\beta_0 = 28.4/d + 2.03$$

$$\beta_t = (p_t)^{\alpha} - 1$$
$$\alpha = (-0.146 / (a/d - 2.93) - 0.978) \qquad (a/d \geqq 3.0)$$

$$\beta_w = 2.70\ (p_w - 0.1)$$

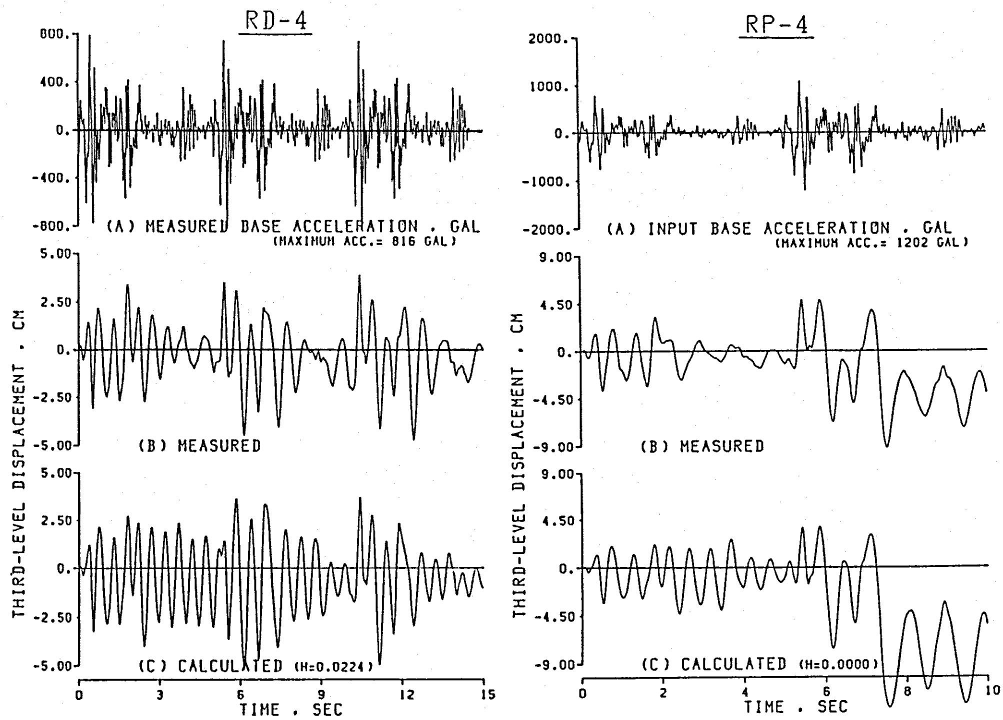
$$\beta_a = \begin{cases} (-0.0153\sigma_0 + 0.175)\ (a/d - 4.0) & (\sigma_0 \leqq 11.4\text{kg/cm}^2) \\ 0 & (\sigma_0 > 11.4\text{kg/cm}^2) \end{cases}$$

$$\beta_N = 2.18\ (\sigma_0 + 10)^{-0.260} - 1$$

$$\beta_n = 1.26\ (n)^{-0.0990} - 1$$

d:effective depth (cm), pt:longitudinal reinforcement ratio (%), a/d:shear span ratio, pw:web reinforcement ratio (%), σ_0:axial compressive stress (kg/cm^2), n:number of repetitions of loading.

Equation(3) was derived from summarizing quantitatively the effects of various factors(effective depth, longitudinal reinforcement ratio, shear span ratio, web reinforcement ratio, axial compressive stress, number of repetitions of loading) on ductility of reinforced concrete members which had sectional characteristics



(1) RD-4 (2) RP-4

Fig.13 Time history of displacement obtained from experiments and analyses using proposed model

similar to ordinary reinforced concrete piers of a single column type.

The stiffness(kd), which indicates the slope of moment to rotational angle after the point U in **Fig.12**, was determined as **Eq.(4)**. **Eq.(4)** was derived from the many test results of the past studies.

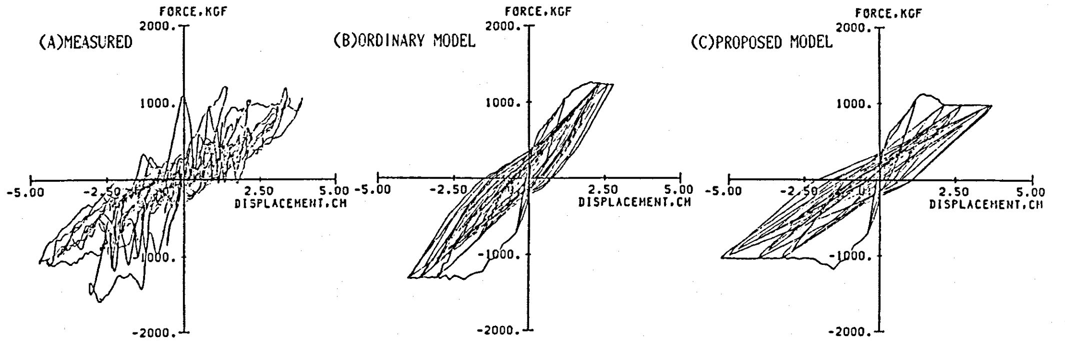
$$(-k_d)/k_y = 1.229(\mu_u - 1)^{-1} - 0.0539 \quad \text{-----(4)}$$

in which ky=the stiffness at yielding(segment C-Y in **Fig.12**). The rule of the hysteresis curve is the same one used in the Takeda's model.

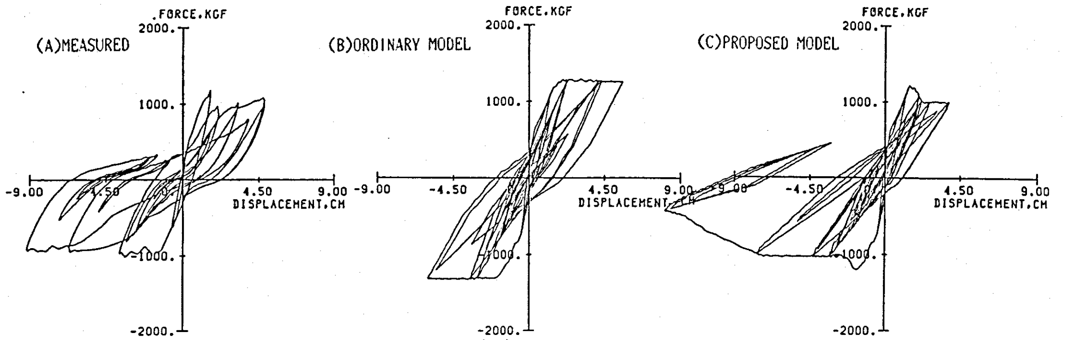
6.2 Response Analyses Using Proposed Restoring Force-Displacement Model

Using the proposed restoring force model, response analyses were carried out for all the test structures. **Figure 13** shows the time histories of the top displacement obtained from the tests and analyses using the proposed model for structures RD-4 and RP-4 which failed in shear after yielding of the longitudinal reinforcements. The maximum response values and the periods of excitation obtained from analyses agree well with those from the tests after shear failure occurred in the first-level beam(after 0.7 sec). That is, the inelastic response behaviors in the ultimate state can be calculated accurately by using the proposed restoring force model even if the load carrying capacity of some member decreased suddenly caused by the occurrence of shear failure.

Figure 14 shows the measured and calculated base shear-displacement response curves. The calculated ones were obtained from the ordinary restoring force



(1) RD-4



(2) RP-4

Fig.14 Base shear-displacement relationship obtained from tests and analyses

model as well as the proposed model. Though the ordinary restoring force model can't represent the response behaviors of the structure in the case that the load carrying capacity of the structure is decreasing due to the occurrence of shear failure in one member, the proposed model can simulate accurately the whole behaviors of the structure up to the failure. The proposed model is also very useful method to predict damage of each member as well as the whole structure.

Recently, it is strongly required to have a sufficient capacity of inelastic deformation without decrease of the load carrying capacity , that is ductility, for reasonable earthquake resistant design of reinforced concrete structures. Generally, in statically indeterminate structures such as reinforced concrete rigid frame piers, the whole structure may not collapse even if some member failed and then lost the load carrying capacity perfectly. However, the inelastic response behaviors of the structure change remarkably dependent on the ultimate failure mode of the member, and a safety for earthquakes also depends on the ultimate properties of the member. Therefore, it is necessary to calculate accurately the response behaviors of the structure after some member yielded and then failed. If this becomes possible, it will be able to obtain quantitatively the ductility of the structure and to predict the extent of damage of the structure and each member during earthquakes. Furthermore, in order to design reinforced concrete rigid-frame piers reasonably as an earthquake-proof structure, the members which have proper strength and ductility should be arranged adequately. The proposed restoring force model is a very powerful method for these problems.

6. CONCLUSIONS

In order to clarify inelastic behaviors of reinforced concrete frame structures subjected to severe earthquakes, simulated earthquake tests and pseudodynamic tests were carried out using small scale two-story one-bay reinforced concrete bridge piers, and inelastic response analyses based on one component model were conducted. It is concluded that;

(1)The inelastic response behaviors of reinforced concrete frame structures changed remarkably after shear failure occurred and then the load carrying capacity decreased in some member. Therefore, in order to design reinforced concrete rigid-frame bridge piers as a seismic resistant structure, beams as well as columns need to have adequate ductility without decrease of the load carrying capacity.

(2)The maximum base shears obtained from the simulated earthquake tests were about 20 % higher than those obtained from the pseudodynamic tests due to strain rate effect of the reinforcing bars of each member. It must be pointed out that the response behavior obtained from the pseudodynamic test may not represent a real behavior of reinforced concrete structures subjected to earthquakes.

(3)The calculated responses based on one component model agreed generally with the measured ones at all levels of excitation during the tests except that the calculated base shear was smaller than the measured one due to strain rate effect. The inelastic response behaviors can be calculated satisfactorily by using the usual restoring force model in the case that all the members, which constitute the structure, would fail in flexure. In the case that load carrying capacity of some member decreased suddenly due to the occurrence of shear failure, however, the inelastic response behaviors can't be calculated sufficiently by the usual restoring force model. Therefore, the new restoring force model which can represent the ultimate characteristics of each member was proposed.

(4)The inelastic response behaviors of reinforced concrete frame structures can be calculated accurately by using the proposed restoring force model even if the load carrying capacity of some member decreased suddenly caused by the occurrence of shear failure. The proposed model is also a very powerful method to predict damage of each member as well as the whole structure.

ACKNOWLEDGMENT

A part of this study was conducted as the joint research by Saitama University and Central Research Institute of Electric Power Industry.

REFERENCES

[1] Japan Road Institute :Specification for Road Bridge, Earthquake Resistant Design, 1980.
[2] Ozaka,Y.:Problems of the Design Method for Reinforced Concrete Structures from the Point of View of the Damages Due to Miyagi-Ken Oki Earthquake, CONCRETE JOURNAL, Vol.17,No.9,1979.9.
[3] Railway Technical Research Report:Report on Miyagi-Ken-Oki Earthquake on June 12,1978, The Railway Technical Research Institute, Japanese National Railways, 1978, No.1111,1979.3.
[4] JSCE:Standard Design for Structures of Japanese National Railway.
[5] Mutsuyoshi,H. and Machida, A.:Behavior of Reinforced Concrete Members Subjected to Dynamic Loading", CONCRETE LIBRARY, No.6, JSCE, 1985.

[6] Mutsuyoshi,H., Machida, A., and Tsuruta, K.:Dynamic Nonlinear Earthquake Response of reinforced Concrete Structures Based on Strain Rate Effect, CONCRETE LIBRARY, No.6, JSCE, 1985.
[7] Okada,T.:Simulation of Earthquake Response of Structures by Computer-Actuator On-Line System, CONCRETE JOURNAL, Vol.20, No.1, 1982.(in Japanese)
[8] Giberson,M.F.:Two Nonlinear Beams with Definitions of Ductility, Proc. of ASCE, Vol.95, No.ST2,1969.
[9] Clough, R.W, Benuska, K.L. and Wilson, E.L.:Inelastic Earthquake Response of Tall Buildings, Proc. of 3rd WCEE, Vol.1,1965.1.
[10] Otani, S.:Inelastic Analysis of R/C Frame Structures, Proc. of ASCE, Vol. 100,No.ST7,1974.7.
[11] Takeda,T., Nielsen,N.N. and Sozen, M.A.:Reinforced Concrete Response to Simulated Earthquakes, Proc. of ASCE, Vol.96, No.ST12.1970.12
[12] Eto, H. and Takeda,T.:Inelastic Response Analysis of R/C Frame Structures, Annual Meeting of Japan Architectual Association,1977.
[13] Wilson, E.L. and Penzien, J. :Evaluation of Orthogonal Matrices, International Journal for Numerical Methods in Engineering, Vol.4, 1972.
[14] Machida, A., Mutsuyoshi, H. and Toyoda, K.:Evaluation of the Ductility of Reinforced Concrete Members, Proc. of JSCE NO.378/Vol.6,1987.2.
[15] Ohta, M.:A Study on Earthquake Resistant Design for Reinforced Concrete Bridge Piers of Single-Column Type, Report of the Public Works Research Institute Ministry of Construction, Vol.153, 1980.