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# INFLUENCE OF DUCTILITY OF MEMBERS ON INELASTIC RESPONSE BEHAVIOR OF REINFORCED CONCRETE FRAME STRUCTURES

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## SUMMARY

In order to clarify inelastic response behaviour of R/C frame structures subjected to earthquake motion, shaking table tests and pseudodynamic tests were carried out using small scale two-story one-bay R/C bridge piers, and inelastic response analyses based on one component model were conducted. Since the inelastic response behaviour of R/C frame structures depended strongly on the capacity of plastic deformation, that is ductility, of each member, it couldn't be calculated accurately using an ordinary response analysis method when shear failure occurred in some member. Therefore, a new restoring force-displacement model which can represent well ductility of each member was proposed. Using the proposed restoring force model, the inelastic response behaviour of R/C frame structures could be calculated with satisfactory accuracy even if some members failed perfectly in shear.

## INTRODUCTION

Many R/C rigid-frame highway and railway bridge piers have been constructed in Japan, and some of them have been damaged due to strong earthquakes. For example, in Miyagi-ken-oki Earthquake of 1978, many columns and beams of R/C rigid-frame piers of elevated railway bridges were remarkably damaged[1]. Generally, in the case of a statically indeterminate structure such as R/C rigid-frame piers, the structure will not collapse even if one of the members which constitute the structure fails perfectly. However, the inelastic response behaviour of the structure may be influenced by failure of the member. In order to design reasonably a R/C rigid-frame structure as an earthquake-proof structure, it is very important to make clear the seismic properties of R/C structures in the plastic range as well as in the elastic one because a design load and safety for earthquakes depend strongly on the inelastic behaviour. The objectives of this paper are to clarify experimentally and analytically the influence of ductility of members on the inelastic response behaviour of R/C frame structures subjected to strong ground motion and to establish the method to calculate the inelastic response behaviour.

## OUTLINE OF EXPERIMENT

The test structures are two-story one-bay R/C frames which are intended to represent a portion of typical bridge.piers used for the elevated railways of the Tohoku-Shinkansen. The test structures were designed assuming that the following failure modes would occur; 1)flexural failure at the bottom of the first-level column(structures RD-1 and RP-1), 2)flexural failure in the first-level beam (RD-3), 3) shear failure 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. In every test, a weight of 963 kgf, which produced the axial stress of 9.6 kgf/cm² in a column, was installed at each top of both second-columns. 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 structure into an inelastic range, the original time scale was compressed by a factor of 2 while the maximum base acceleration was amplified to 0.8*G. A general view of the test set-up is shown in Fig.1. The pseudodynamic tests were carried out to investigate in detail the reponse behaviour at each step. The test method proceeded statically in stepwise manner under a step by step integration procedure. 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), the whole system was assumed as a single degree-of-freedom. The pseudodynamic system is shown in Fig.2.

Table 1 Details of 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).

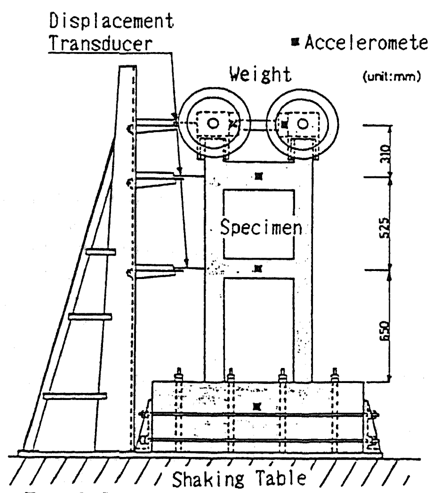



Fig.1 Test set up for simulated earthquake test

## RESPONSE ANALYSIS BASED ON ORDINARY RESTORING FORCE MODEL

In order to obtain analytically the response behaviour of the structures, member-by-member analyses based on one component model[4] 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. 3. 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(KpA and KpB:spring constant). The shear deformation of a member is represented by the elastic shear spring(Ks:spring constant) at the center of the member. Takeda's model[5] and Takeda's slip[6] model were used for columns and beams as restoring force models, respectively.

Figure 4 shows the measured and calculated time histories of the base shear and the top displacement for the structure RD-3 of which the first-level beam failed finally in flexure. The calculated responses agree generally with the measured ones during the tests. The analytical model is very available for the frame structures on condition that all members fail in flexure.

Figure 5 shows the measured and calculated time histories for RD-4 whose first-level beam failed in shear. The period of the calculated responses is clearly shorter than that of the measured ones after shear failure occurred(1.0 sec) in the first-level beam. Figure 6 shows the measured and calculated base shear-displacement curves for RD-4. The measured base shear and stiffness of the hysteresis curves become small remarkably as compared with the calculated ones after shear failure occurred in the first-level beam. These results indicate that the

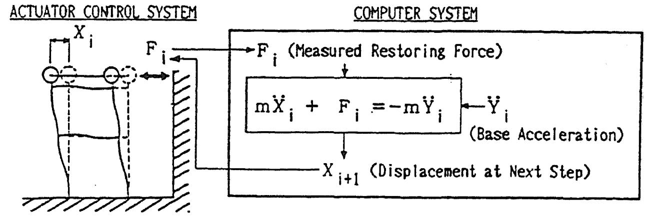
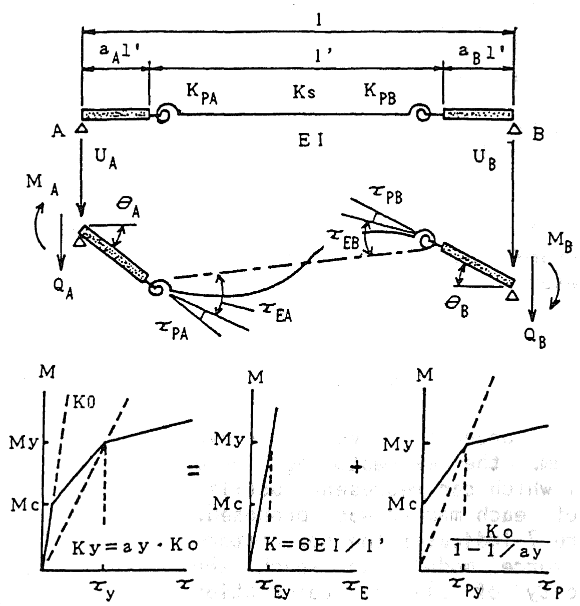



Fig.2 System for pseudodynamic test




$$\tau = \tau_E + \tau_P$$

Fig.3 Concept of one component model

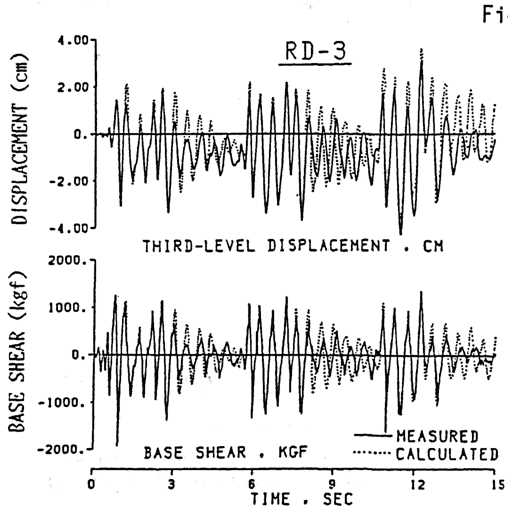



Fig.4 Displacement and base shear obtained from test and analysis(RD-3)

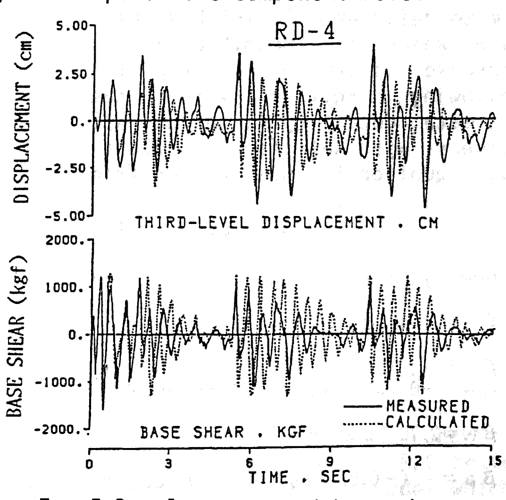



Fig.5 Displacement and base shear obtained from test and analysis(RD-4)

ordinary restoring force model can't simulate accurately the inelastic response behaviour of the structure after shear failure occurred in some member and then the load carrying capacity of the member decreased suddenly. This fact also shows the limitation of the used model. Generally, in a statically indeterminate structure such as R/C rigid-frame piers, the inelastic response behaviour of the structure depends strongly on the failure mode, that is the capacity of plastic deformation, of members even if the structure may not collapse. Therefore, the restoring force model which can represent well ductility of all members is required to calculate precisely inelastic responses in all plastic ranges.

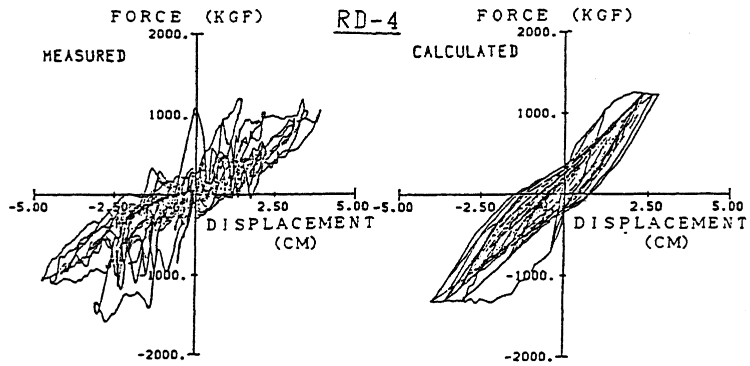



Fig.6 Measured and calculated base shear-displacement relations(RD-4)

## RESPONSE ANALYSIS BASED ON DUCTILITY OF MEMBERS

In order to solve the above problem, the new restoring force model which can represent ductility of each member was proposed. Figure 7 indicates the new restoring force model, in which the capacity of plastic deformation and the characteristics that the strength of a member decreases are taken into consideration. The ultimate deformation(point U in Fig. 7) at which the strength begins to decrease is determined from the following Eq.(1) which can estimate ductility factor of R/C members with satisfactory accuracy. Equation (1) was derived from

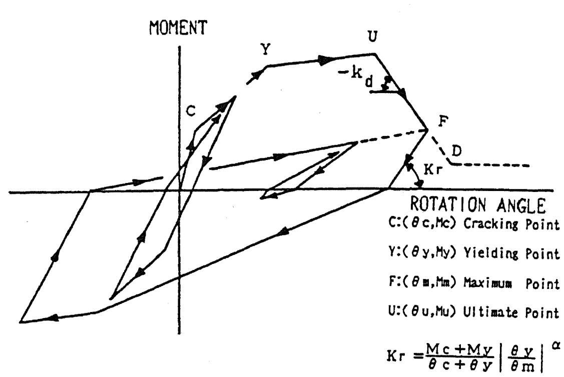



Fig.7 Proposed restoring force model considering ductility of member

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

where, $\mu_u$ :ductility factor(ultimate displacement / yielding displacement)

$\beta_0 = 28.4/d + 2.03$

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

$\beta_w = 2.70 (p_w - 0.1)$

$\beta_a = (-0.0153\sigma_0 + 0.175)(a/d - 4.0)$

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

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

d:effective depth (cm)
pt:longitudinal reinforcement ratio (%)
a/d:shear span ratio
pw:web reinforcement ratio (%)
$\sigma_0$:axial compressive stress (kg/cm²)
n:number of repetitions of loading

summarizing quantitatively the effects of various factors, such as effective depth, longitudinal reinforcement ratio, shear span ratio, web reinforcement ratio, axial compressive stress and number of repetitions of loading, on ductility of R/C members[7]. The slope after the point U is defined by Eq.(2) which was derived from many test results. The hysteresis rule of the proposed model is the same one as used in the Takeda's model.

$$(-k_d)/k_y = 1.229(\mu_u - 1)^{-1} - 0.0539 \quad \text{-----}(2)$$

in which, ky=stiffness at yielding(segment C-Y in Fig.7).

Using the proposed restoring force model, response analyses were carried out for all test structures. Figure 8 shows the time histories of the top displacement obtained from the tests and analyses for structure RP-4 whose first-level beam failed in shear after yielding of the longitudinal reinforcements. The response values and the periods of excitation obtained from the analysis agree well with those from the tests after shear failure occurred in the first-level beam(after 1.0 sec). That is, the inelastic response behaviour can be calculated accurately by using the proposed restoring force model even if the strength of some member decreased suddenly caused by the occurrence of shear failure.

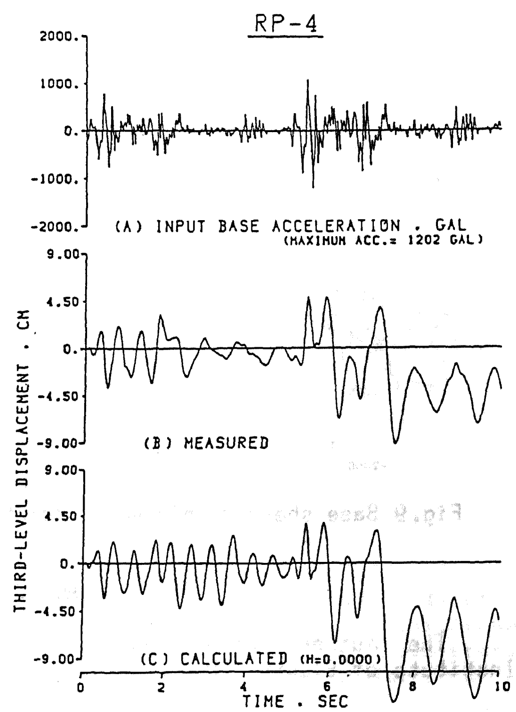



Fig.8 Time history of displacement obtained from test and analysis using proposed model

Figure 9 shows the measured and calculated base shear-displacement response curves. The calculated ones were obtained from both the ordinary model and the proposed one. Though the ordinary model can't represent the response behaviour accurately after the load carrying capacity of the structure decreased due to the occurrence of shear failure in one member, the proposed model can simulate the whole behaviour of the structure up to the failure with satisfactory accuracy. Moreover, the proposed model can be a powerful method to predict the extent of damage of each member as well as a structure.

## CONCLUSIONS

Since the inelastic response behaviour of R/C frame structures depends strongly on the capacity of plastic deformation, that is ductility, of each member, it can't be calculated accurately using an ordinary response analysis method when shear failure occurs in some members and the load carrying capacity of the members decreases. In order to obtain the response behaviour of R/C frame structures in all plastic ranges, a new restoring force-displacement model which can represent well ductility of each member was proposed. The inelastic response behaviour of R/C frame structures could be calculated with satisfactory accuracy using the proposed restoring force model even if some members failed perfectly in shear.

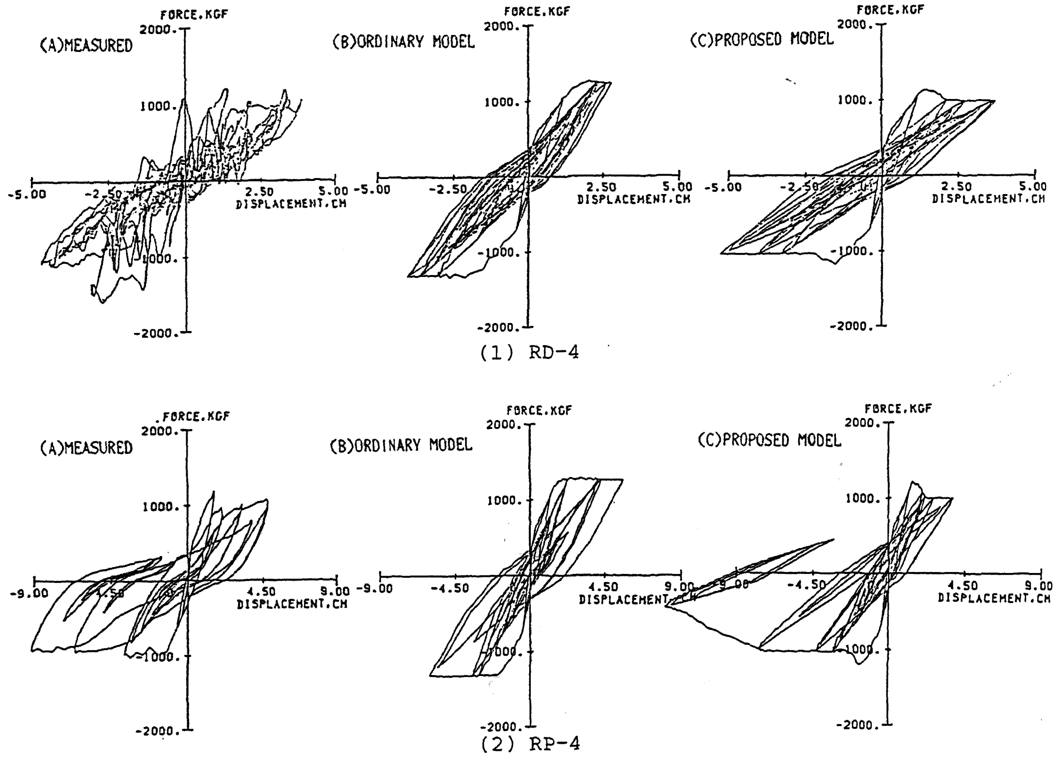



Fig.9 Base shear-displacement relations obtained from tests and analyses

## ACKNOWLEDGMENT

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