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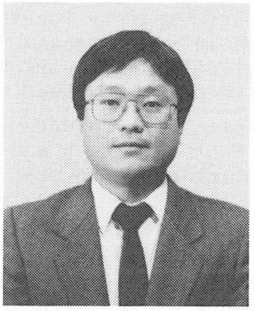
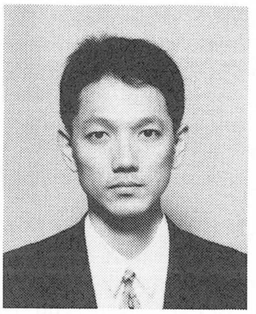
RESPONSE OF REINFORCED CONCRETE COLUMNS SUBJECTED TO EARTHQUAKE FORCES WITH RELATION TO THE EVALUATION IN SEISMIC DESIGN

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SYNOPSIS

In order to investigate the response characteristics of a reinforced concrete column under earthquake forces and to evaluate the seismic design method in JSCE's Standard Specification of Concrete in 1986, static and pseudo-dynamic tests using reinforced concrete column specimens and related analyses were carried out. It was confirmed that the classification of the level of serviceability after the earthquake stipulated in the Specification was adequate from the obtained cracking pattern of columns subjected to reversed cyclic loadings. From the results of pseudo-dynamic test, the appropriateness of the limit state modification factor specified in the Specification was verified. The response of a reinforced concrete column under an actual earthquake which caused definite shear failure was presented properly by pseudo-dynamic test. Analytical method to evaluate the static inelastic displacement and the seismic response of reinforced concrete columns were proposed and were confirmed to be adequate.

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1. INTRODUCTION

A new seismic design method based on limit state design was stipulated in Chapter 9 of JSCE's Standard Specification of Concrete in 1986 [1]. The peculiarity of the concept of the design method is such that seismic design should be performed to fulfill required serviceability after the design earthquake as well as required safety during the earthquake. High magnification factor due to dynamic response was introduced according to actual observation in the earthquakes, and reduction factor referred to the acceptable level of damages remained in the structure after the earthquake was adopted in the design method. The limit state under a design earthquake should be determined in relation to necessary strength and deformability which are based on safety and serviceability during and after the earthquake, respectively[2].

Therefore, for a rational design of a structure subjected to earthquake forces, it is important to evaluate the nonlinear behavior of the structure under the design earthquake. Although experimental and analytical studies had extensively been carried out on the response of the structure under earthquake by many researchers, the nonlinearities under earthquake were not sufficiently clarified because the nonlinear behavior was influenced by many factors such as the shape of cross section, the ratio of tensile reinforcement, axial force, the shear span-depth ratio, the characteristics of the earthquake and so forth. Moreover, it is difficult to predict analytically the response of reinforced concrete structures failing in shear.

In the present study, static and pseudo-dynamic tests[3] using reinforced concrete column specimens and related analyses were carried out in order to investigate the response characteristics of reinforced concrete columns subjected to earthquake forces including shear failure and to evaluate the seismic design method of JSCE in 1986. In the analyses, analytical models to evaluate the static inelastic displacement and the seismic response of reinforced concrete columns were proposed. The results obtained from the analytical models were compared with the experimental results.

2. OUTLINE OF EXPERIMENTAL WORK

The experimental study consists of two series. Series I is to confirm the appropriateness of the classified four damage levels occurred after the earthquake and of the limit state modification factor ν_4 which were given in the Specification. Reversed cyclic loading test and the pseudo-dynamic test were used in this study. The pseudo-dynamic test is an experimental technique in which the dynamic response of the specimen subjected to a seismic wave is obtained in combination with on-line computer analysis. In the Specifications, there are four levels of damage according to the maximum response displacement in such a way that $1\delta_y$ (here, δ_y is yield displacement) displacement corresponds to sound condition, $2\delta_y$ displacement corresponds to light damage, $3\delta_y$ displacement corresponds to medium damage and $4\delta_y$ displacement corresponds to significant damage. However, the actual damage of the reinforced concrete structures corresponding to each level of damage stipulated in the Specification had not yet been realized clearly. Therefore, in series I, the cracking patterns for each displacement were observed in detail using reinforced concrete column specimens(hereafter RC columns) subjected to static reversed cyclic loadings. In addition, the obtained results of cracking patterns and load carrying capacity were discussed in order to confirm the appropriateness of the damage classification, i.e., the serviceability after the earthquake given in the Specification.

The limit state modification factor ν_4 is a reduction factor for the earthquake forces according to the serviceability of the structure after the earthquake. The appropriateness of each level of ν_4 is discussed by comparing the calculated design value with the maximum response displacement obtained from pseudo-dynamic test subjected to actual earthquake forces corresponding to each ν_4 [4].

On the other hand, series II aimed to present definitely the response of an RC column failing in shear under an earthquake. In series II experiment, pseudo-dynamic test was carried out using RC columns in which the amount of the hoop reinforcement was equal to 30% of the required value[5].

3. SPECIMENS

Fig.1 shows the details of the specimens of both series. The characteristics of each specimen used for both series are similar except for the diameter and spacing of the hoop reinforcement. D13 bars (13 mm diameter deformed bars) are used for longitudinal reinforcement and the ratio of the reinforcement is 3.8%. D3 bars which were made exclusively for experiments is used for hoop reinforcement of series I specimen. The ratio of hoop reinforcement is 0.35%, the spacing and the amount of which is 2cm and is equal to the required value of JSCE's Specification of 1980 edition [6]. Series I has six specimens. Two specimens were used for static test and other four specimens were used for pseudo-dynamic test.

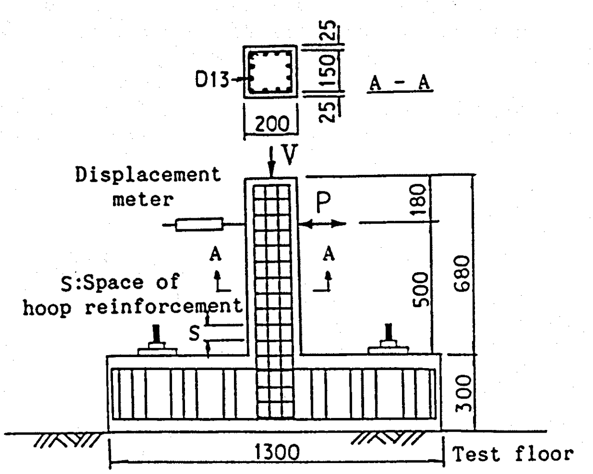


Fig.1 - Details of specimen and loading condition

Table 1 - Arrangement of hoop reinforcement

Specimen	Size	Space (cm)	Ratio (%)
Series I	D3	2	0.35
IID type	D6	6	0.53
IIB type	D3	6	0.12

Table 2 - Mechanical properties of materials

Size	Type	Yield strength	Tensile strength	Application
Reinforcement				
		N/mm²		
D13	SD30	363	553	Series I
D13	SD30	353	550	Series II
D3	SD30	329	492	Series I
D3	SD30	324	496	IIB type
D6	SD35	422	552	IID type
Concrete				
Maximum size of aggregate		Compressive strength (N/mm^2)		
13mm		39		Series I
13mm		26		Series II

Series II has two types of specimen. The first specimen type (hereafter IIB(brittle) type) is designed to consider shear failure. On the other hand, the second specimen type (hereafter IID(ductile) type)is designed to consider bending failure to compare with IIB type specimens. D3 and D6 hoop reinforcements are used in IIB type and IID type, respectively and the spacing of hoops is 6cm in both type. The ratios of the hoop reinforcement for IIB type and IID type are 0.12% and 0.53%, whereas their amounts are equal to 30% and 140% of the necessary value of JSCE's Specification of 1980 edition. The number of specimen are five where three of them are IIB type and the other specimens are IID type. Each one of both type specimens was used for static test and remaining three specimens were used for pseudo-dynamic test. Table 1 shows the arrangement of the hoop reinforcement in each specimen. Table 2 shows the mechanical properties of the materials used in the both series.

4. LOADING AND MEASURING METHOD

Each specimen was fixed to the test floor by using PC tendon to prevent the moving of the specimen as shown in Fig.1. The loads of static and pseudo-dynamic test were applied using two actuators. One actuator applied a constant axial compressive force at the top of the column, and the other actuator applied horizontal force or displacement. In order to investigate the influence of the axial force, a 39.2 kN axial force ($V/Ac=0.98\ N/mm^2$, where V is axial compressive force and Ac is area of cross section in column) was applied to one specimen of series I static test, and 98.1 kN axial force ($2.45\ N/mm^2$) was applied to another specimen of series I static test. The axial force of other specimens of both series was 39.2 kN which corresponds to dead load in actual structures.

The pattern of static reversed cyclic loading was as follows: After applying axial force, horizontal force was introduced in one complete cycle of the yield load Py. Here, Py is a load at which the tensile stress in the outside longitudinal reinforcement reaches its yield value. Then, displacement control method was used such as one cycle of $2\delta_y$ displacement (δ_y is horizontal displacement under the yield load), one cycle of $3\delta_y$ and so forth until the specimen failed.

An earthquake wave of El-Centro 1940 (NS) which ranged from 0.02 to 8.0 seconds was used in the pseudo-dynamic test of both series. The initial conditions in the pseudo-dynamic test were as follows: The damping ratio was 0.05. The

Table 3 - Maximum acceleration and modification factor

Level of damage		L.S.M. factor (ν_4)	Maximum accele. (m/s^2)	
			Series I	Series II
$1\delta_y$	Sound condition	1.0	0.22	0.27
$2\delta_y$	Light damage	0.7	0.31	0.38
$3\delta_y$	Medium damage	0.55	0.40	0.49
$4\delta_y$	Significant damage	0.4	0.55	0.67
Failure level			——	1.34

L.S.M. : Limit state modification δ_y : Yield displacement

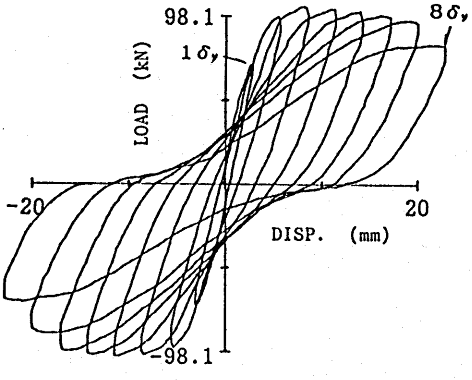
vibration mode was one degree of freedom. In order to carry out the pseudo-dynamic test at large acceleration response in El-Centro earthquake, the natural period of the structure was assumed to be 0.5sec. and the virtual lumped mass of series I and series II were 1.56 MN and 1.39 MN, respectively. Here, the virtual lumped mass is the value which was calculated using the secant modulus at the yield loads of load-displacement curves obtained from static test. Table 3 shows the maximum acceleration which was applied to specimen and the limit state modification factors ν_4. These accelerations were adjusted to correspond with the values of ν_4. In series II test, the acceleration which was two times larger than the value of significant damage level (hereafter failure level) was applied to the specimen to obtain the severe failure. Each of four specimens in series I was loaded by the acceleration corresponding to each level of four damages. In the case of series II, each type of specimen was loaded starting from the level of the light damage. Only one specimen of IIB type was loaded starting from the level of significant damage.

Observations of crack pattern in series I under static reversed cyclic loading were carried out at every maximum displacement and also after unloading. In this case, the crack pattern were sketched out and photographs were taken at a distance of 80 cm from the surface of the specimen without writing the crack lines on the surface of the specimen by pen. The displacement at the position of loading point was measured continuously using the X-Y recorder.

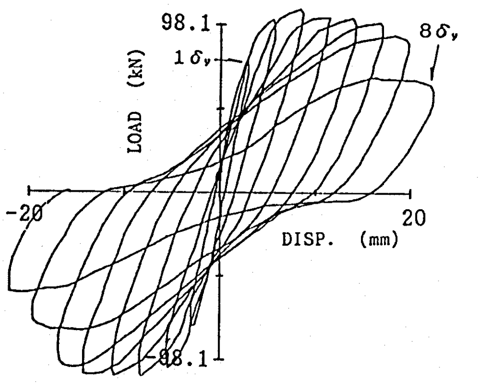
5. EXPERIMENTAL RESULTS AND DISCUSSIONS

5.1 Series I

Fig.2(a) and (b) show the relationship between the horizontal load and displacement for specimens with 39.2 kN and 98.1 kN of applied axial forces under static loading, respectively. In the case of 39.2 kN axial force, the measured displacement under the yield load was 2.85 mm which was referred to the yield displacement ($1\delta_y$) in series I. The maximum load carrying capacity was 103 kN. The ratio of the residual unloaded displacement to the maximum displacement increased according to the increasing of the maximum displacement and it was approximately 0.5 under $4\delta_y$ loading. The load carrying capacity under $8\delta_y$ loading was about 70% of the maximum value because of the growth of the diagonal cracks. On the other hand, in the case of 98.1 kN axial force, the load under $1\delta_y$ loading was 76.5 kN and the maximum load carrying capacity was 107 kN. Both the stiffness and the maximum load carrying capacity were slightly larger than those of the specimen with 39.2 kN axial force. Moreover, the growth of the diagonal cracks was slow in the specimen having 98.1 kN axial force. However, when the width of the diagonal cracks became large, load carrying capacity decreased and under $8\delta_y$ loading it decreased to about 60% of the maximum load carrying capacity.



(a) Axial force (V) : 39.2 kN



(b) Axial force (V) : 98.1 kN

Fig.2 - Horizontal load - displacement curves
(Series I)

Fig.3 shows the observed crack pattern of the specimen with 39.2 kN axial force under static loading. The cracks were found initially when their width became 0.05 mm. When the width of cracks increased to 0.2 mm, cracks could be seen at a distance of about 80 cm from the surface of the specimen. The maximum width of the crack under $1\delta_y$ loading which corresponds to sound condition was 0.3mm and this crack could not be found after unloading. The first remaining crack after unloading could be found when the loading reached $2\delta_y$ displacement which corresponds to light damage. However, the maximum width of remaining crack was below 0.1 mm. The maximum crack width reached to 1.5 mm under $4\delta_y$ which corresponds to significant damage and the maximum width of the remaining crack after unloading was only 0.2 mm to 0.4 mm. The diagonal crack grew remarkably after the $5\delta_y$ loading. From these observations, the specified classification of damage level was found to be adequate.

The load-displacement curves don't show clearly the yielding at the calculated yield load as shown in Fig.2(a) and (b). The reason is due to the existence of longitudinal bars in the web portion and also the negligence of the influence of the reaction point at the root of the column which should be considered in the calculation of load carrying capacity (refer to 6.1 and Fig.7).

Fig.4 shows a comparison between maximum response displacements obtained from the pseudo-dynamic test and the design value. Here, these specimens were designed using ν_3 value as 2.0 which is a response modification factor in the Specification of 1986 edition. Furthermore, the pseudo-dynamic tests were carried out under the acceleration response spectrum equal to 2.5 corresponding to the natural period of 0.5 sec. in El-Centro earthquake wave as mentioned above. As shown in Fig.4, the measured experimental response values approximately coincide with or less than the design values in all the damage levels. Therefore, the specified values of the modification factors ν_4 in the Specification are in safe side and are satisfactory for intended design.

The crack width corresponding to each damage level obtained from pseudo-dynamic test was obviously less than that obtained from static test because the obtained response displacements were less than the design displacements. The remaining crack width was very small in each damage level after the loading test. The maximum remaining crack width under significant damage was only 0.2mm to 0.4 mm.

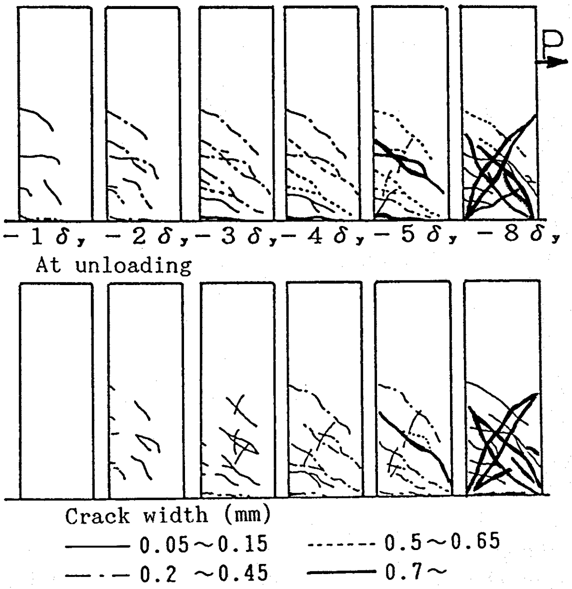


Fig.3 - Cracks in the columns under static loading at various displacements (Constant axial force (V) : 39.2 kN)

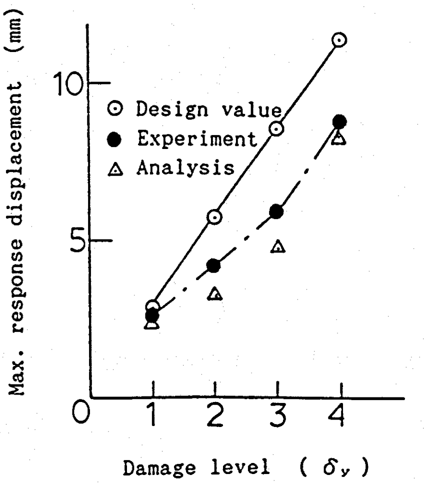
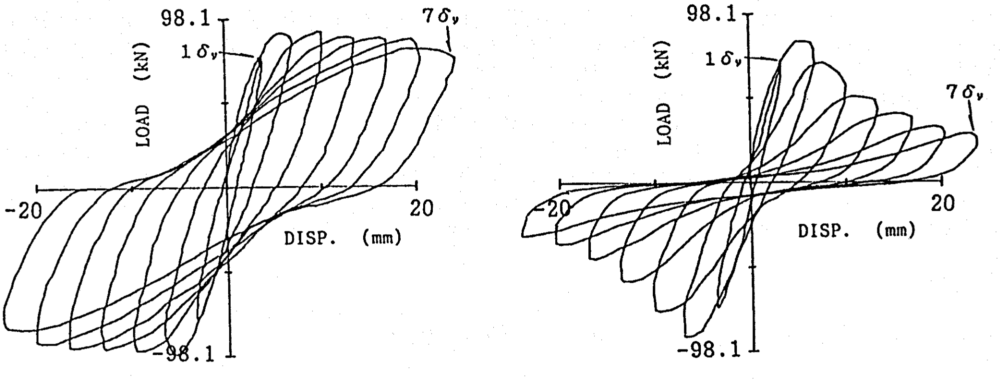


Fig.4 - Max. response displacement vs. design value

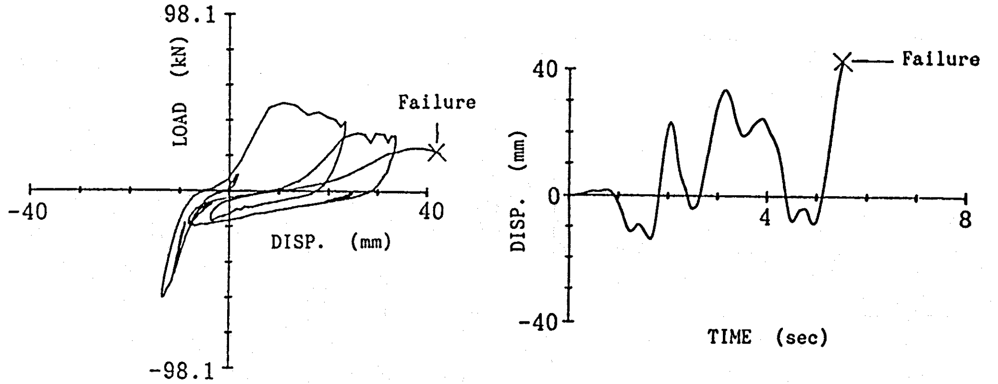
5.2 Series II

Fig.5(a) and (b) show the relationship between horizontal load and displacement for IID type and IIB type specimens under static loading, respectively. In the case of IID type where the specimen had sufficient amount of hoop reinforcement, the displacement under the calculated yield load was 3.38 mm which is correspondent with the yield displacement in series II. IID type specimen showed ductile behavior and the remaining load carrying capacity under $7\delta_y$ was about 80% of the maximum load carrying capacity. On the other hand, in the case



(a) IID type (b) IIB type

Fig.5 - Horizontal load - displacement curves (Series II)



(a) Load - response displacement curve

(b) Response displacement

Fig.6 - Behavior of IIB type specimen under the earthquake of failure level

of IIB type specimen whose amount of hoop reinforcement was insufficient, the diagonal cracks extended largely during the load of 3 δ_y and the load carrying capacity decreased abruptly. That is, the load carrying capacity under 7 δ_y was about 30% of the maximum load carrying capacity. In IIB type specimen, the load carrying capacity increased until 2 δ_y loading despite the extreme lack of the hoop reinforcement. The reason of this phenomenon is due to the contribution of longitudinal bars in web portion for shear resistance.

As shown in Fig.5, the load-displacement curves don't indicate clearly the yielding at the calculated yield load as same as that of series I due to the longitudinal bars in web portion. However, in series II, the influence of the reaction point at the root of the column was considered in the calculation of the yield load(refer to 6.1 and Fig.7).

Fig.6(a) and (b) show the load-response displacement curve and the response displacement at the failure level obtained from the pseudo-dynamic test of the IIB type specimen which was loaded starting from the level of significant damage. Since the stiffness of the IIB type specimen was reduced abruptly due to the extension of the diagonal cracks, the natural period of response displacement became extremely longer than the initial natural period and also the response displacement was increased. Finally, the specimen failed by an increment in the response displacement during the loading as illustrated in these figures. Since the pseudo-dynamic test can predict the dynamic response of structures without knowing the nonlinear behavior of the structures, pseudo-dynamic test is quite useful for the prediction of response displacement of the structures whose stiffness changed largely such as the structures failing in shear.

6. ANALYSIS OF LOAD CARRYING CAPACITY AND DISPLACEMENT

6.1 Method of Analysis

A fiber model in which the cross section was replaced into layerd fiber elements was adopted in the calculation method. The displacements of the RC columns were obtained by integrating the relationship of moment-curvature of the cross section along the longitudinal direction.

In order to calculate the static inelastic displacement easily and accurately, one-dimensional model considering the rotation at the root of column was proposed as shown in Fig.7 [7]. To evaluate the rotation at the root of column, the joint portion of the column and the footing is assumed to be D/2(here, D is height of the cross section) below the surface of the footing. In other words, it is assumed that the D/2 length of column is extended inside the footing. this phenomenon has been regarded as " bond slip" of the longitudinal bars from the footing by other researchers[8]. However, the concept of "bond slip" is not comprehensive because it is influenced largely by the diameter of the longitudinal bars. Furthermore, since the column and the footing are replaced into one-dimensional members, the reaction point is assumed to be D/4 above the surface of the footing as shown in Fig.7 in accordance with the two dimensional effect.

The shear displacement was considered in an analysis. As shown is Fig.8, the direction of the principal stress is assumed to be 45 degrees and the shear displacement in Δla is indicated as $\Delta\delta_v$. Here, the shear displacement δ_v can be expressed by the stress of the hoop reinforcement σ_{sw} as shown in Equation(1).

$$\delta_v = 2\sigma_{sw} \cdot la/Es \qquad (1)$$

Where, la: shear span length
Es: Young's modulus of the hoop reinforcement

The stress of the hoop reinforcement can be expressed by Equation(2). The design shear stress f_{vcd} of the concrete is applied corresponding to the equation based on the limit state design method in JSCE's Specification of 1986 edition.

$$\sigma_{sw} = (S - \alpha \cdot f_{vcd} \cdot b_W \cdot d) \cdot s/(A_S \cdot j \cdot d) \qquad (2)$$

Where, S: shear force
α: modification factor
d: effective depth
j·d: arm length

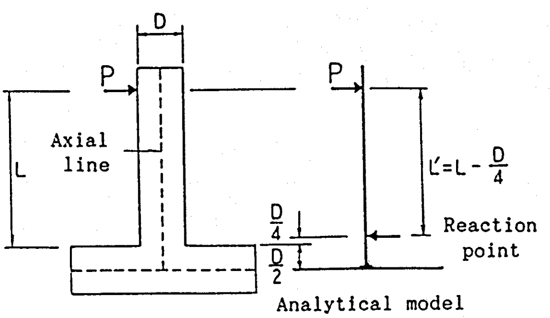


Fig.7 - Analytical model of RC column with footing

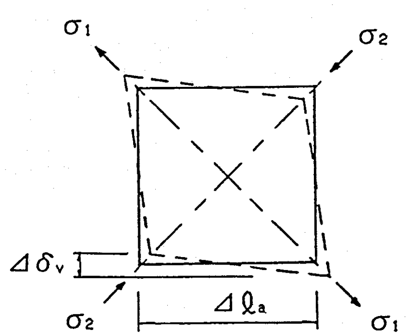


Fig.8 - Shear displacement

bw: width of cross section
s: spacing of shear reinforcement
As: area of shear reinforcement in s
f_{vcd}: design shear stress of the concrete

In this calculation, the modification factor α is assumed to be 1.5 since the value of f_{vcd} includes the safety due to the design equation[9].

6.2 Analytical Results and Discussion

Fig.9(a) and (b) show the comparison between the envelope curves of load-displacement obtained from the static test and the calculated values for both series I specimen with 39.2kN applied axial force and IIB type specimen under shear failure, respectively.

In these figures, the calculated values in which the fixed point of the column is assumed to be at the surface of the footing without considering the rotation is shown by a dotted line. This curve differs significantly from the experimental value, especially after the yielding of the longitudinal reinforcement. On the other hand, the calculated values using the analytical model shown in Fig.7 coincide well with those obtained from experiments. Furthermore, the calculated values coincide very well with experimental value when shear displacement is considered in the calculation. The calculated displacement values considering shear displacement show slightly larger than those obtained from the experiments, especially in IIB type specimen. It can be said that the reason of this result is due to the contribution of the longitudinal bars in web portion for shear resistance as mentioned before. In the case that 15% of the longitudinal reinforcement in web portion is assumed to be added as hoop reinforcement, the calculated values coincide well with the experimental values as shown in Fig.10(a),(b). Here, the calculation in Fig.10(b) was stopped when the strain of hoop reinforcement reached to the yield strain. From these results, the adequateness of the proposed one-dimensional model for RC column was confirmed.

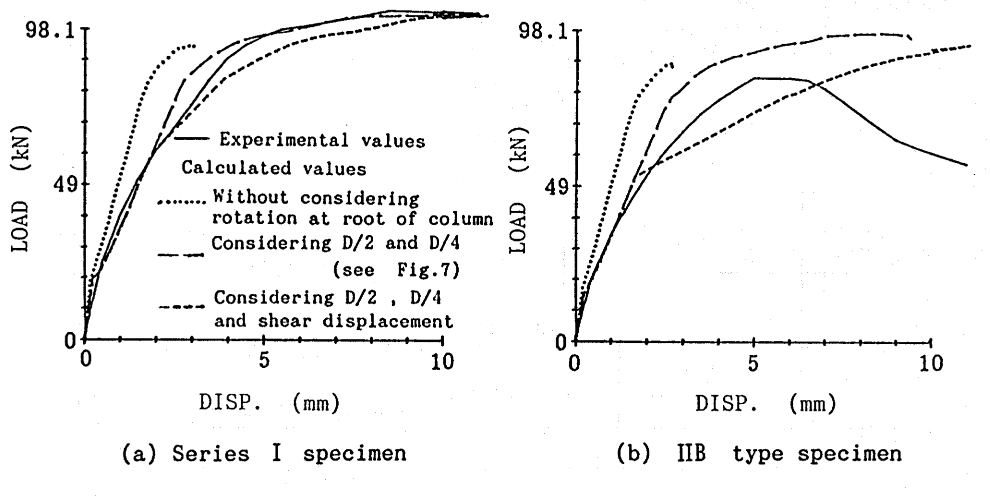
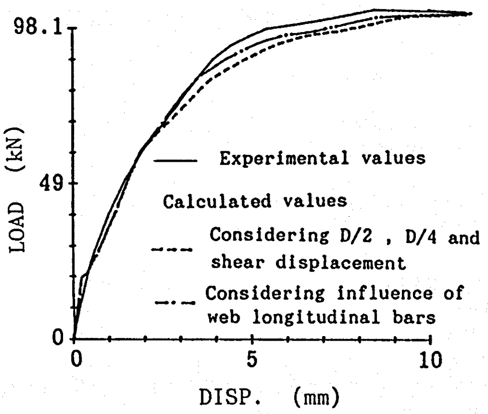
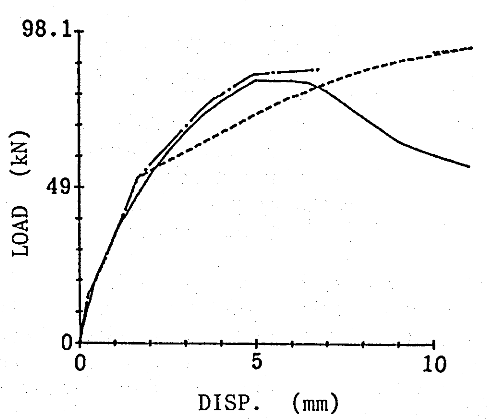


Fig.9 - Conparison between envelope curves of experimental values and calculated values under static loading (1) (Axial force (V) : 39.2 kN)



(a) Series I specimen



(b) IIB type specimen

Fig.10 - Comparison between envelope curves of experimental values and calculated values under static loading (2) (Axial force (V) : 39.2 kN)

7. RESPONSE ANALYSIS

7.1 Analytical Hysteresis Loop Model

Fig.11 shows the analytical hysteresis loop model used in the response analysis. The model was proposed on the basis of the ordinary "stiffness degradation model" in which the unloading stiffness after the yielding was decreased according to the increment of the plastic ratio of the member. The features of the proposed model are that the skeleton of the envelope curve can be expressed by many straight lines and that the stiffness under unloading can be divided into two parts such as high loading stage and low loading stage. Also, since it is possible in this model to change the decrement ratio of the stiffness according to the number of the loading cycles, this model can express

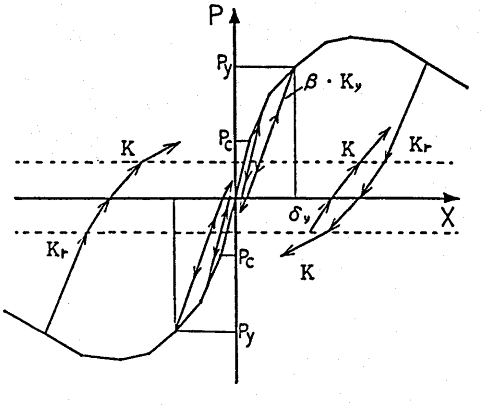


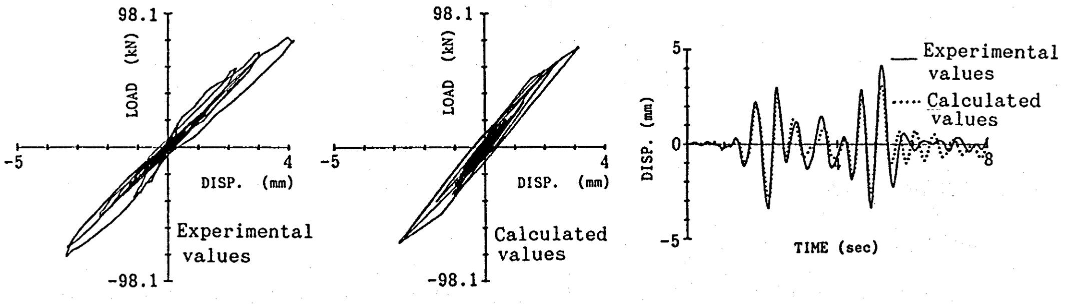
Fig.11 - Model of hysteresis loop

degradation due to shear failure. The unloading stiffness after the yielding, Kr, can be expressed by Equation (3).

$$K_r = \beta \cdot K_y \cdot (\delta_m / \delta_y)^{(-\alpha)} \qquad (3)$$

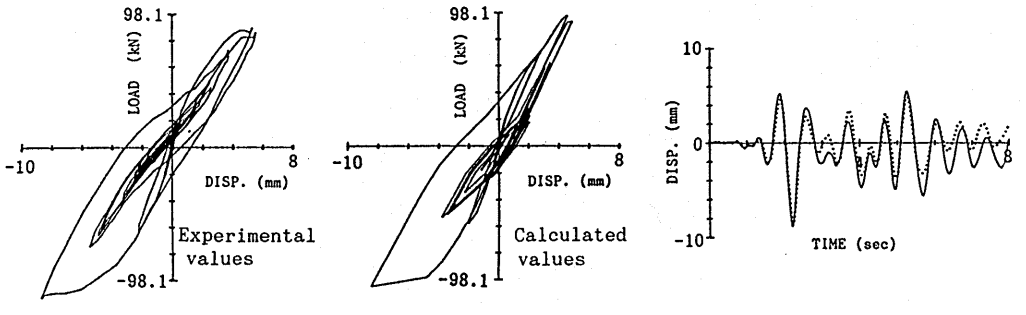
Where, Ky: secant stiffness at the yield load
δ_m: maximum displacement in the past
δ_y: yield displacement
α: decrement factor of stiffness under unloading after the yielding
β: residual displacement factor

The stiffness K under reloading is the secant modulus toward the past maximum displacement before the yielding. After yielding K is obtained by multiplying secant modulus with the decrement factor of stiffness under reloading. All factors used in this model were determined based on the static test results. The calculation was carried out using linear acceleration method. The damping ratio was 0.05 as same with the case of pseudo-dynamic test.



(a) Load - response displacement (b) Response displacement

Fig.12 - Comparison between experimental values and calculated values under $2\delta_y$ level (Series I)



(a) Load - response displacement (b) Response displacement

Fig.13 - Comparison between experimental values and calculated values under $4\delta_y$ level (Series I)

7.2 Results and Discussion of the Response Analysis

Fig.12(a) and (b) show a comparison between the experimental values and the calculated values for both the load-response displacement curve and response displacement-time curve in series I specimen subjected to the earthquake force of the light damage level. Fig.13(a) and (b) show the same comparison for series I specimen subjected to the earthquake force of the significant damage level. As shown in these figures, the calculated values of the maximum displacement and the response period coincide well with those of experimental values. As shown in Fig.13(b), the experimental values of the response displacement of specimen subjected to the earthquake force of the significant damage level shifted toward the negative side after six second of earthquake wave and the calculated values shifted toward the positive side. The reason of this difference may be due to the influence of the deterioration occurred during many reversed cyclic loadings in low level loadings which was not considered in this calculation. However, the fact that the response displacement of a specimen subjected to the earthquake force of significant damage level coincide well with the calculated value may mean that the specimen keeps its integrity after the earthquake of significant damage level.

Fig.14 shows a comparison between the experimental values and the calculated values of the response displacement under significant damage level in IIB type specimen whose shear reinforcement is insufficient. The calculated values of the ductile type don't coincide with the experimental values after the extension of the diagonal cracks. On the other hand, the calculated values with modified stiffness according to number of the loading cycle designated as brittle type coincide well with the experimental values. In this calculation, the stiffness after second cycle under the same level of loading was assumed to be 90% of the stiffness of the first cycle at low loading stage after the yielding. This ratio was obtained from the experimental results. From these results, it was shown that the response displacement for the specimen under the shear failure can be calculated by modifying the stiffness according to the number of the loading cycles.

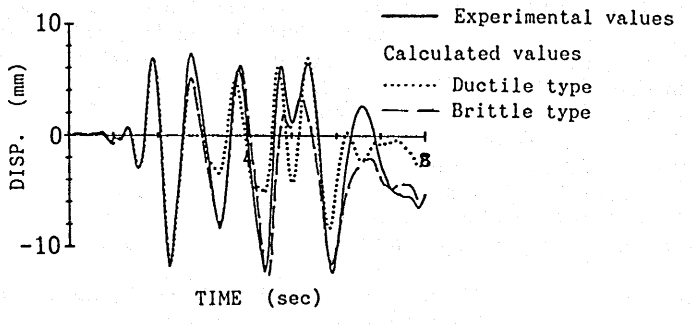


Fig.14 - Comparison between experimental values and calculated values of response displacement under $4\delta_y$ level
(IIB Type specimen)

8. CONCLUSION

Following conclusion can be drawn within the scope of this experimental and analytical study.

(1) In the case of the specimen in which the amount of shear reinforcement was sufficient, it was confirmed that the classification of the level of serviceability after the earthquake stipulated in the JSCE's Specification of 1986 edition was adequate from the experimental results of the cracking pattern and the load carrying capacity of columns subjected to reversed

cyclic loadings. Moreover, the decrement of the load carrying capacity was not found before $4\delta_y$ loading which is corresponding to the significant damage level.

(2)From the results of pseudo-dynamic test, the appropriateness of the limit state modification factor ν_4 specified in the JSCE's Specification of 1986 edition was verified for El-Centro earthquake wave.

(3)The ongoing shear failure phenomenon of RC column under actually recorded seismic acceleration was observed. The pseudo-dynamic test is quite useful to realize the ongoing seismic failure because the phenomena is shown slowly.

(4)The displacement and load carrying capacity of an RC column with footing can be calculated properly by using the proposed analytical model. The calculated values using this model coincide well with the experimental values in which the adequateness of the analytical method considering shear displacement was confirmed.

(5)It was confirmed from the experimental results that the longitudinal bars in web portion contribute to the resistance of the shear force as if 15% of these volume would be equivalent to the volume of hoop reinforcement.

(6)The response displacement under actually recorded seismic acceleration can be calculated by using proposed hysteresis loop model in which the various factors were determined based on the results of the static test. Also, the response displacement for the specimen failing in shear can be calculated by modifying the stiffness corresponding to the loading cycle.

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