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Study on the formation of plastic hinges and the failure of reinforced concrete columns

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ABSTRACT: Main purpose of this study is to make clear the formation process of plastic hinges at the end portions of reinforced concrete columns caused by mainly shear load and the failure process of the member by experiment and analysis. Experimental study on reinforced concrete columns loaded by both axial load and horizontal cyclic load is carried out. During the process to the ultimate state of a member, the effects of axial load and detailing of tie reinforcements are discussed being based on the mechanism of plastic hinges taking into account the analytical study by the finite element method. As the result of this study, followings are obtained. After bending and shear cracks at the end portions of a member at the first stage of yielding, for further horizontal displacement of the member, plastic hinges at the end portions are formed by bending and shear cracks. The final stage is reached when shear failure at the end portions of the member is caused.

1.Introduction

To realize earthquake resistant reinforced concrete structures, columns are necessary to be ductile enough to make sure the structures should not fail in brittle state under earthquake shear loading.

For reinforced concrete columns, ductility depends much on the way of forming plastic hinges at the end portions due to shear load.

In former studies (1), (2), the failure properties of reinforced concrete short columns loaded by shear reversals which are affected by the magnitude of axial loading, tie ratios, shear span ratios, etc. were investigated.

In this study, experimental study using 10 specimens of reinforced concrete short columns of shear span ratio of 2.5 and analytical study using the finite element method are carried out.

2.Experimental Study

2.1 Experimental Program

For all the column specimens, the cross section is 20cm x 20cm, the length is 100cm, the ratio of shear span to depth is 2.5 and the main bar ratio is 1.0%. The configuration of the specimens is shown in Fig.1. The variables which are considered to affect the behavior of reinforced concrete columns subjected to axial load(N) and shear load(Q) are as follows:

(1)Axial load(N):
 10tonf(25kgf/cm^2), 30tonf(75kgf/cm^2)

(2)Tie ratio(Pw):
 Pw=0.56%-@50mm
 Pw=0.56%-@110mm
 Pw=0.85%-@75mm
 Pw=1.28%-@50mm
 Pw=1.28%-@100mm

The region of the end portions is 20cm length region from the member ends and the middle portion is the central 60cm length region as shown in Fig.1. Table 1 shows the list of test specimens. Specimen names are given in the order of [axial load (tonf)]-[tie ratio (%), tie pitch @}(mm)].

Table 1. List of specimens.

10-0.56@50	10-0.85@75	10-1.28@50
30-0.56@50	30-0.85@75	30-1.28@50
10-0.56@110		10-1.28@100
30-0.56@110		30-1.28@100

The loading set-up is shown in Fig.2. The loading process is as follows: (a) to apply constant axial load, (b) to apply horizontal shear load of 3tonf and 6tonf, (c) to apply horizontal shear load to cause first visible bending crack, (d) to apply horizontal shear load to cause first visible shear crack, (e) to apply 4 cycles of shear load for relative horizontal displacement of ±10mm (deflection angle of 1/100), (f) to apply 3cycles of

Table 2. Experimental results.

	Strength of concrete (kgf/cm²)		Shear Load (tonf)											
			Initial			Displacement/length								
Specimen	Compression	tension	bending cracking		Maximum	at 1/100		at 2/100		at 3/100		at 4/100		
10-0.56@ 50	370.0	29.4	3.0	-2.0	-8.7	7.5	-7.0	8.4	-8.7	8.2	-8.3	6.9	-7.1	
30-0.56@ 50	370.0	29.4	5.0	-5.0	11.6	10.4	-9.5	10.8	-10.3	9.0	-8.7	7.7	-6.8	
10-0.56@110	224.4	11.3	4.0	-5.0	7.4	5.9	-6.1	7.4	-7.4	6.7	-6.5	5.7	-5.2	
30-0.56@110	219.2	16.1	6.3	-6.8	8.5	7.8	-7.1	8.5	-7.6	6.0	-4.7	-	-	
10-0.85@ 75	224.4	11.3	4.1	-4.1	7.6	6.3	-5.4	7.6	-7.0	6.8	-6.3	5.8	-5.9	
30-0.85@ 75	219.2	16.1	7.0	-6.5	-8.2	6.7	-7.4	7.9	-8.2	6.6	-7.9	3.9	-4.5	
10-1.28@ 50	224.4	11.3	4.3	-4.5	7.5	6.8	-6.4	7.5	-7.4	7.2	-7.1	6.2	-6.4	
30-1.28@ 50	219.2	16.1	5.5	-7.0	-8.3	7.5	-7.5	8.1	-8.3	7.0	-7.4	6.6	-6.9	
10-1.28@100	370.0	29.4	3.0	-3.0	8.4	6.6	-6.8	7.7	-7.8	7.4	-7.1	6.7	-6.9	
30-1.28@100	301.3	28.5	6.5	-5.5	11.1	9.6	-9.4	10.7	-10.7	9.1	-9.2	7.8	-7.0	

shear load for relative horizontal displacement of ±20mm (deflection angle of 2/100), ±30mm (3/100) and ±40mm (4/100) and (g) to apply 1cycle of shear load for relative horizontal displacement of ±50mm (deflection of 5/100) as shown in Fig.3.

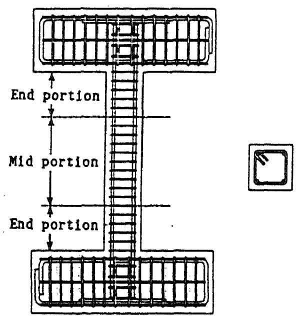

Figure 1. Test specimens.

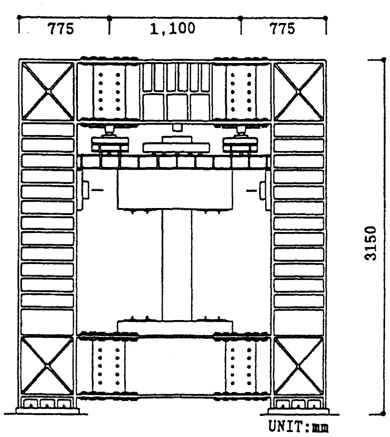

Figure 2. The loading set-up.

Horizontal and vertical displacements are measured by dial gages. Strains in main steel bars and steel ties are measured by wire strain gages.

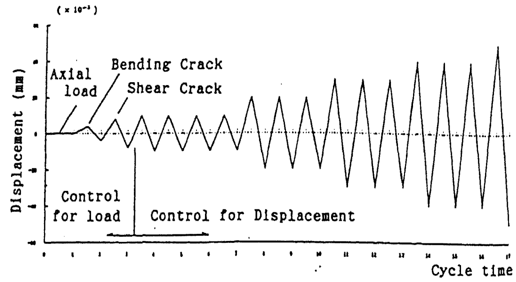

Figure 3. Loading process.

2.2 Result of Experiment

Table.2 shows experimental result. Figure 4 shows typical shear loading and deformation relationship and Fig.5 shows some typical crack patterns for four deflection angles.

Specimens under higher axial load are more brittle than those under lower axial load as seen from Fig.4. Specimens with lower tie ratios are more brittle than those with higher tie ratios as seen from the same figure.

As seen from Fig.5, crack patterns show that initial bending cracks occur near the member ends for all specimens. The following bending cracks occur at further portions from the ends and these portions become nearer to the middle portion as the axial load become smaller. The following bending-shear cracks occur at further portions from the ends.

As for the effect of axial load, the columns under 30tonf axial loading fail in brittle state after the maximum shear loading due to the cracks along the axial direction which are caused by concrete crash due to bending and bending-shear cracks and it is observed that columns under higher axial loading are more brittle than those under lower axial loading.

As for the effect of tie ratios, the columns with lower tie ratios fail in brittle state after the maximum shear loading due to broadening of concrete crash zones and the

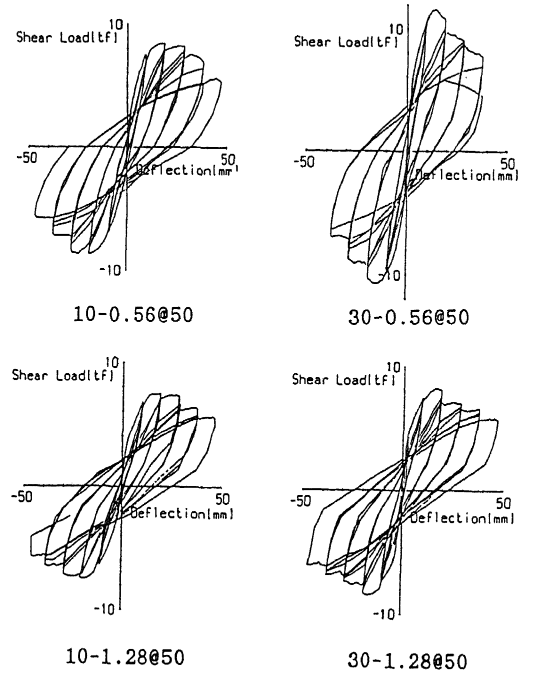

Figure 4. Typical shear loading and deformation relationship.

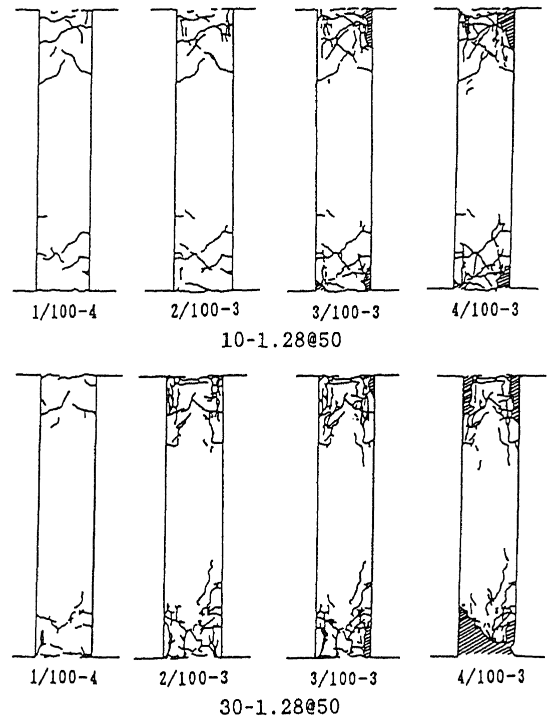

Figure 5-b. Typical crack patterns.

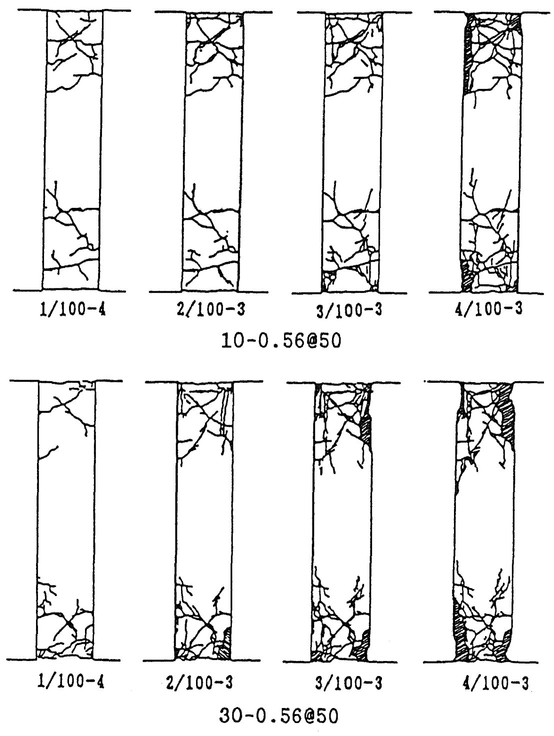

Figure 5-a. Typical crack patterns.

zones of concrete separation from main bars and it is observed that these columns are more brittle than those with higher tie ratios.

3.Analytical Study

3.1 Analysis by the Finite Element Method

The finite element method is applied for the analytical study. The analytical model is shown in Fig.6 and this model is adopted after the actual reinforced concrete specimens which are used in the experiment.

Concrete is represented by constant strain triangular elements . For the behavior of concrete under biaxial stress , the modified incremental orthotropic model , which was proposed originally by Darwin and Pecknold(5) being based on uniaxial strain and was modified by Noguchi et al.(6) who took into account the effect of rotation of principal axes due to shear stress, is used. Material properties of concrete are shown in Table.3.

Tensile steel bars are represented by one-dimensional element. Ties are represented by smeared model. The stress-strain relationship of steel is bilinear type and material properties of steel are shown in Table.3.

To express the stress and bond slip relationship between tensile steel and concrete, bond link element is used. Material properties of bond link are shown in Table.3. Cracks of concrete are expressed by using smeared crack model.

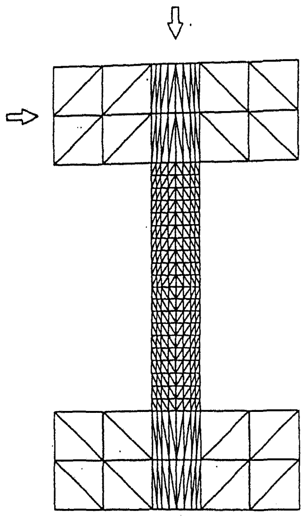

Figure 6. Analytical model.

Table 3. Material constant.

(a) Concrete	
Uniaxial compressive strength, f'_c (kgf/cm²)	220.0
Uniaxial tensile strength, f'_t (kgf/cm²)	25.0
Strain for f'_c,	0.003
Initial elastic modulus, Ec (kgf/cm²)	222000.0
Poisson's ratio ν,	0.42
(b) Steel(main bar, tie bar)	
Yield strain,	0.0019
Initial elastic modulus, Es (kgf/cm²)	2050000.0
Modulus after yielding, Esy (kgf/cm²)	41000.0
(c) Bond link	
Initial elastic modulus, Eb (kgf/cm³)	5500.0
Modulus after yielding, Eby (kgf/cm³)	4500.0
Yield (cm),	0.0005

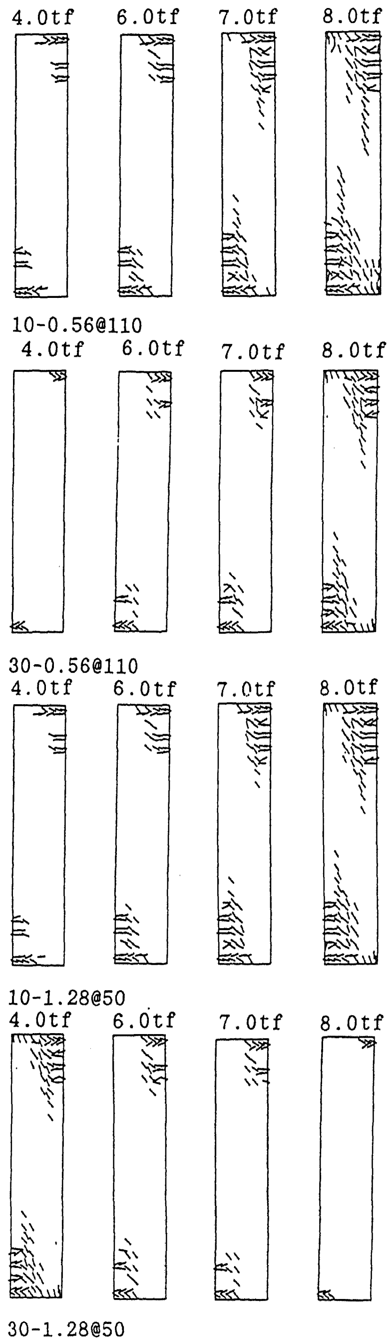

Figure 7. Crack distribution.

3.2 Result of Analysis

As seen from Fig.7, the cracks occur in the order of bending cracks, bending-shear cracks in the end portions and bond splitting cracks for all specimens and this cracking process is nearly equal to the experimental results. It is investigated first about the effect of axial loading. The first bending crack for the specimen under 10tonf axial load occurs near the end portions at 2.5tonf shear load, whereas for the specimen under 30tonf axial load the first bending crack occurs at 3.25tonf shear load.

For the specimen under 10tonf axial load, bending-shear cracks occur in the portions at 15-20cm from the ends, whereas for the specimen under 30tonf axial load the portions where bending-shear cracks occur shift to the portions at 10-15cm from the ends.

The cracks which occur in the middle portion of the specimens are not many for both cases, but the cracking portions are wider for the specimen under 10tonf axial load than for the specimen under 30tonf axial load.

As for the effect of tie ratios, as the ratios become lower, the end portions of the specimens where cracks occur become wider for the case of 10tonf axial load than for the case of 30tonf axial load.

In Figs.8 and 9, the strain distributions in main steel bars and in tie steel bars are shown respectively. At shear load near 4tonf, tensile force is caused in main bars and this force becomes larger as cracks develop as shear loads become larger than 6tonf and at this stage compressive force also start to increase. As seen from Fig.8, near maximum shear load of 8tonf, the main bars in tension are in yielding and the member end rotations become larger. Also tkhe main bars in compression is in yielding due to the crash of concrete elements near the member ends.

These phenomena become more pronounced as the axial loads become lower and the tie ratios become lower.

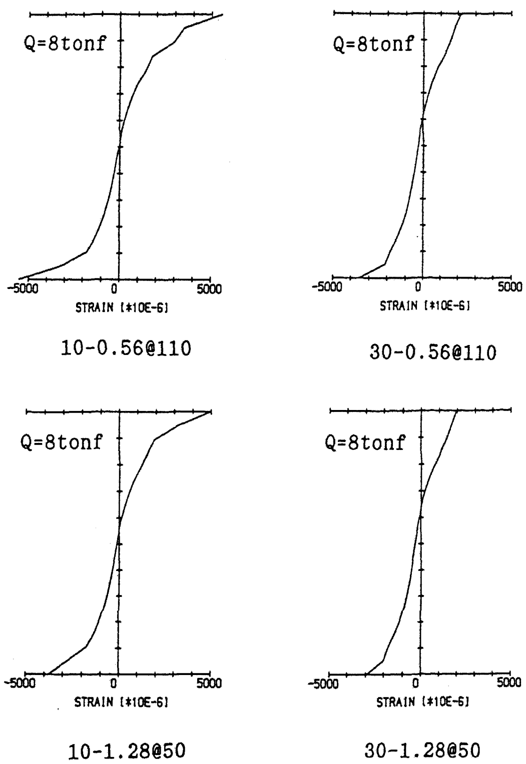

Figure 8. Strain in tensile steel.

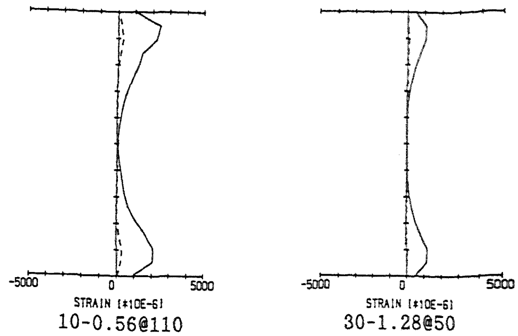

Figure 9. Strain in tie.

For the strains in tie bars, tensile strains become larger as shear load becomes larger and as cracks near the member ends develop. The rate of increase of tie strains becomes larger as bending-shear cracks develop near the member ends after 4tonf shear load for the specimen under the 10tonf axial load and after 7.5tonf shear load for the specimen under 30tonf axial load.

4. Discussion

Figures 10 through 12 show relationships between shear loads and deflection angles. The ordinate shows the ratio of shear loads and the maximum shear load. The shear loads are those for the 3rd cycle of loading for deflection angles of 2/100 and 3/100 and for the 2nd cycle of loading for deflection angle of 4/100.

Figure 11 shows the effects of tie ratios.

Figure 12 shows the effects of tie pitches.

From these figures it is clear that higher tie ratios, denser tie pitches are effective for higher ductility of the member because

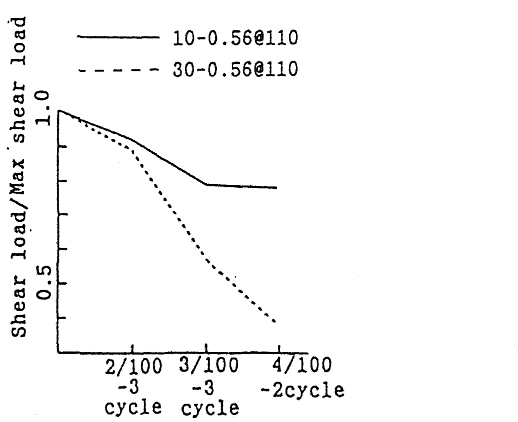

Figure 10. Decay of shear resistance by loading cycle for axial load

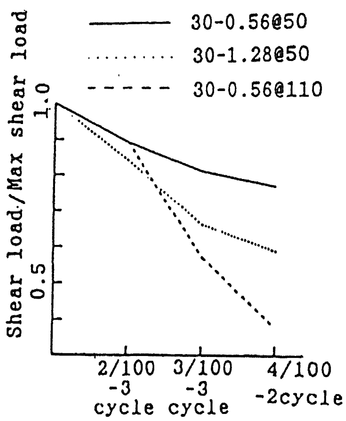

Figure 11. Decay of shear resistance by loading cycle for tie ratio

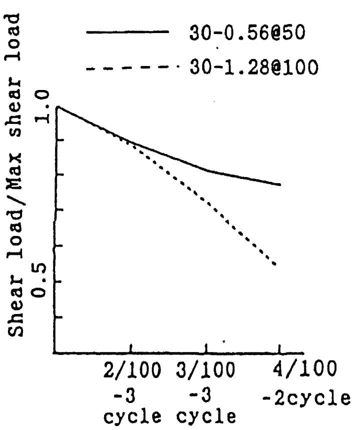

Figure 12. Decay of shear resistance by loading cycle for tie pitch

of the formation of plastic hinges at the member ends due to confining effect for concrete by ties.

5. Conclusion

As the results of this study, following results are obtained. The process of failure of reinforced concrete columns are as follows. At the first stage of yielding, bending and shear cracks occur at the end portions of the member. At this stage, the member would keep ductility still. For larger horizontal displacement of the member, plastic hinges at the end portions are formed by bending and shear cracks at the end portions. The final stage is reached when splitting bond failure at the middle portion of the member is caused or when shear failure at the end portions of the member is caused.

Columns with rich tie reinforcement are effective for preventing ductile failure and also this is so even for the middle portion of the member. For higher axial force, columns are more brittle and this fact should be taken into consideration for practical design.

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