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POST YIELD OVERSTRENGTH OF REINFORCED CONCRETE COLUMNS WITH STEEL TUBES FOR BAR SPLICING IN YIELD HINGE REGION

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ABSTRACT

In the seismic structural design of a reinforced concrete frame system, an idealized model which has flexural yield hinges at every beam end in each story and at the column bottom of the first story is often planned to resist against the seismic action and to make the plastic energy dissipate there. To ensure the strategy planned the strength deformation relationships of the yield hinge should be accurately estimated as possible. Such the design concept is also applied to a precast reinforced concrete frame system. In the precast frame system, steel tubes are often used to splice the longitudinal bars of the column. As for the tube splicing technique, it is known that a considerable overstrength after flexural yield generates due to the early strain hardening of the bars spliced in the yield hinge. The effect of overstrength has to be reflect for the estimation of the design shear force of the columns to ensure the sufficient ductility. This paper presents the behavior of the strength deformation relationships after flexural yield obtained by the laboratory tests and analysis, and the technique how to estimate the overstrength.

KEYWORDS

Reinforced concrete; Precast concrete; Bar splicing; Strain hardening; Overstrength; Design shear force.

INTRODUCTION

In the seismic structural design of a reinforced concrete frame system, an idealized model which has flexural yield hinges at every beam end in each story and at the column bottom of the first story is often planned to resist against an earthquake action and to make the plastic energy dissipate there. To ensure the tactics planned the strength deformation relationships of the yield hinge should be accurately estimated as possible. Such the design concept is also applied to a precast reinforced concrete frame system.

In the precast frame system, steel tubes are often used to splice the longitudinal bars of the column. As for the tube splicing technique, however, some laboratory tests (Matsumori et al., 1993 and Ohkubo, 1994) show that a considerable load increase beyond the flexural strength of the member generates due to the early strain hardening of the bars spliced in the yield hinge. The increase of the strength after flexural yield as the member drift increases is called "post yield overstrength" in this paper. The effect of overstrength has to be reflect in the estimation of the design shear force of the columns to ensure the sufficient ductility.

The flexural strength of a reinforced concrete column is controlled by the combination of parameters; yielding strength of the longitudinal steel and the steel ratio, the pattern of bar placing in the cross-section, and the amount of axial load. An overstrength of the longitudinal steel after the yielding stress also influences the load deformation relationships of the column after the flexural strength.

The flexural strength itself of a reinforced concrete column can be easily predicted by a calculation based on the plastic theory. But a laboratory test may give a significant information about the strength after flexural yielding, particularly for the behavior subjected to load reversals.

In this paper the behavior of the strength deformation relationships after flexural yield of the reinforced concrete columns is discussed on the basis of the laboratory tests. And the technique how to estimate the overstrength is discussed.

TEST PROGRAMS

Test Specimens

Five reinforced concrete columns with/without tube splices for the longitudinal bars were tested with load reversals in inelastic ranges after the flexural yield. The main parameter of the specimens was the axial load level (3.92MPa, 1.96MPa, and 0) as shown in Table 1. The test specimen was a cantilever type of column with the 20cm width and the 40cm depth as shown in Fig. 1. Two longitudinal bars were placed as the middle layers in the column. The tube splices were set at the bottom of the column. Deformed bars of 16mm diameter (D16) were used for the longitudinal steel. The yield strength was 374MPa. The compressive strength of the concrete was 21.6MPa. The non-shrinkage grout mortar was injected into the steel tube to splice the longitudinal bars, and the compressive strength of the mortar was 65.4MPa. The property of the material used is summarized in Table 2.

Table 1. Parameter of the test specimens

Specimen	No. 1	No. 2	No. 3	No. 4	No. 5
Tube splices	with	with	with	without	without
Axial stress (MPa)	0	1.96	3.92	0	3.92

Loading and Measuring

The lateral load reversals were applied to the column at the height of 100cm from the top of the foundation using servo-actuator. The axial load was applied at the top of the column using an oil jack, and it was

Table 2. Mechanical properties of steels (fy: yield strength, Es: Young's modulus, ε_{sh}: strain at strain hardening, Esh: stiffness after strain hardening, ft: tensile strength)

Steel bars	fy (MPa)	Es (GPa)	ε_{sh} (%)	Esh (GPa)	ft
Longitudinal bar (D16)	374	172	2.27	4.83	539
Hoops (5.5mm)	290	193	2.38	3.32	378

Table 3. Mechanical properties of concrete (fc': compressive strength, ε_B: strain at compressive strength, Ec: secant modulus of concrete)

	fc' (MPa)	ε_B (%)	Ec (GPa)
Concrete	21.9	0.23	21.9
Grout Mortar	65.4	0.28	24.3

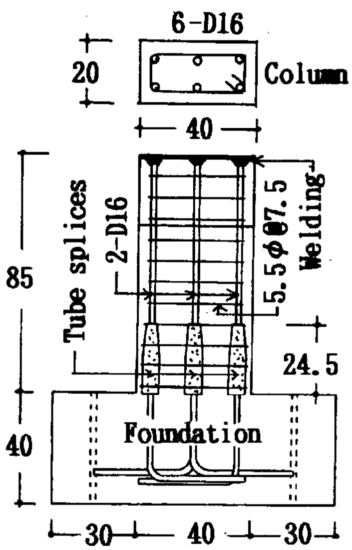

Fig. 1. Dimension and reinforcement of test specimens (unit:cm).

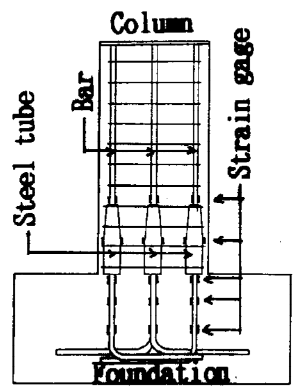

Fig. 2. Location of strain gages for bars.

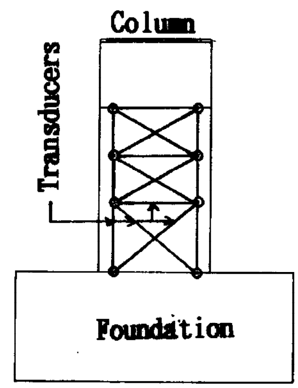

Fig. 3. Arrangement of transducers.

controlled constantly during the test as the corresponding stress shown in Table 1.

The lateral deformation of the column was measured by the electric transducer at the lateral loading point. The strains of the longitudinal steels were measured at the points shown in Fig. 2. The deformation of the sections shown in Fig. 3 were measured using displacement transducers.

LOAD DEFORMATION RELATIONSHIPS

The load deformation relationships of the specimens No. 1, No. 2, No. 3 and No. 5 by the tests are shown in Fig. 4 through Fig. 7, respectively. The extreme tension bars at the bottom of the column yielded approximately at the 0.5 percent of the drift angle. Then the longitudinal bars of the middle layer yielded approximately at the 1.0 percent of the drift angle. The lateral load increase appeared after the flexural yielding of the middle layers in the specimens with the tube splices. The ratio of the load increase reduces as the axial load increases.

The extreme compression bars buckled during the load reversals at the 4 percent of the drift angle in the specimen without the tube splices and with the axial compressive stress of 3.92MPa, and the remarkable lateral load reduction appeared. The ductile load deformation relationships, however, was observed during the lateral load reversals at the 4 percent of the drift angle in the specimen with the tube splices and with the axial stress of 3.92MPa. This indicates that the steel tubes to splice longitudinal bars improved the buckling strength of compression bars.

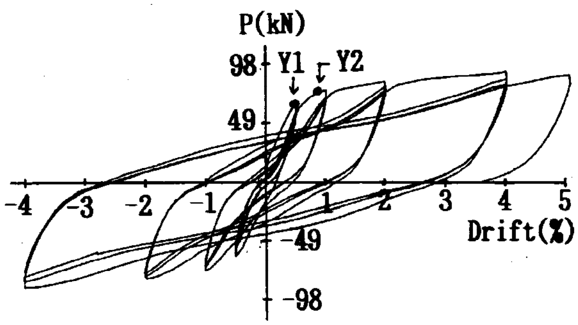

Fig. 4. Load drift relationships of No. 1 Specimen (with steel tubes, axial stress: none).

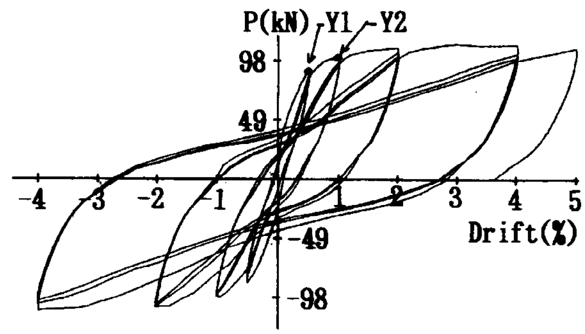

Fig. 5. Load drift relationships of No. 2 Specimen (with steel tubes, axial stress: 1.96MPa).

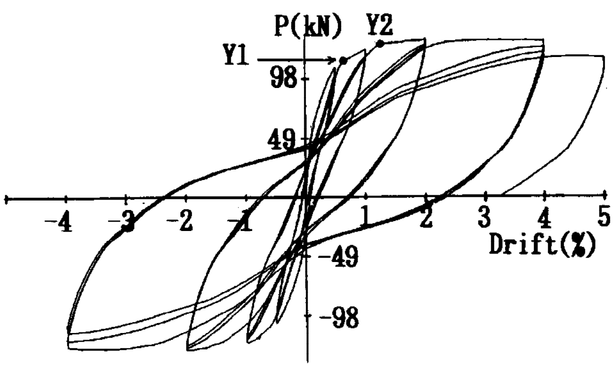

Fig. 6. Load drift relationships of No. 3 Specimen (with steel tubes, axial stress: 3.92MPa).

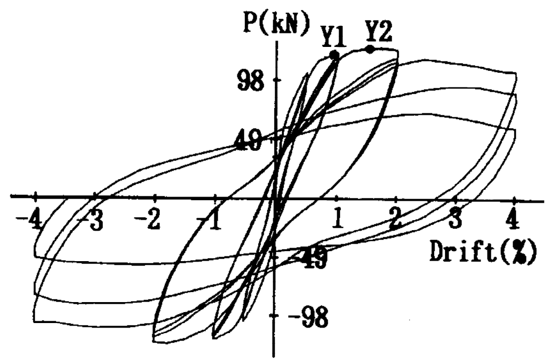

Fig. 7. Load drift relationships of No. 5 Specimen (without tubes, axial stress: 1.96MPa).

STRESS STRAIN MODEL OF STEEL TUBE SPLICES

The tension tests of reinforcing bars spliced by steel tubes were carried out to obtain the stress strain relationships of the bars. The specimen was shown in Fig. 8. The deformation of the bars and the tube, and the displacement of the bars slipped out of the tube were measured in the sections L1 through L5 shown in Fig. 8. The nominal stress strain relationships of the tube obtained by the tests were shown in Fig. 9. The stress(vertical axis) was defined by dividing the tension load by the bar area. The strain(horizontal axis) included the elongation of the tube itself and the displacement of the bars slipped out.

A bilinear stress strain model was assumed for the steel tube on the basis of Fig. 9 to analyze the load displacement relationships of the column. The initial stiffness was 178GPa, and the second stiffness was the four percent of the initial one. The tube itself does not yield, but the nominal yielding stress, which was dominated by the slip out deformation of the bar, was the same as the yielding stress of the bar. The increase of the tensile stress due to the strain hardening was included into the second stiffness in this model.

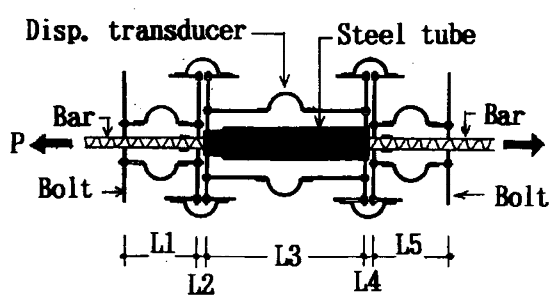

Fig. 8. Tension test set-up of the bars spliced by steel tube.

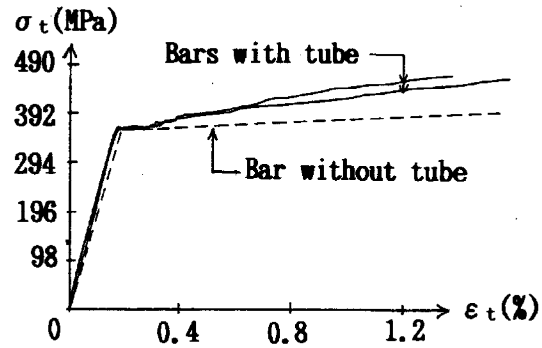

Fig. 9. Stress strain relationships of the bars spliced by steel tube.

ANALYSIS OF LOAD DEFORMATION RELATIONSHIPS OF THE COLUMNS WITH TUBE SPLICES

Analytical Models and Assumptions

The three sections along the height of a column were defined as shown in Fig. 10. The section A is identified as the anchorage part of the tensile reinforcement. The elongation of the bars anchored in the section A makes a rotation at the bottom of the column, and it reflects a corresponding lateral displacement δ_A at the loading point as shown in Fig. 11.

The section H shown in Fig. 10 is identified as the yield hinge zone. The length is the same as the length of steel tube. The flexural deformation of the zone reflects a corresponding lateral displacement δ_H at the loading point as shown in Fig. 12. The curvature in the yield hinge zone is computed on the basis of an inelastic flexural theory using the stress strain relationships shown in Fig. 9 for the steel tubes . A parabolic equation is assumed for the inelastic stress strain relationships of concrete subjected to compression.

The section C shown in Fig. 10 is identified as the ordinal reinforced concrete column. The flexural deformation in the section reflects a corresponding lateral displacement δ_C at the loading point shown in Fig. 13. The curvature in the section C is also computed on the basis of the inelastic flexural theory using the material properties shown in Tables 2 and 3.

The displacement of the column subjected to the lateral load, δ, is estimated by;

$$\delta = \delta_A + \delta_H + \delta_C \tag{1}$$

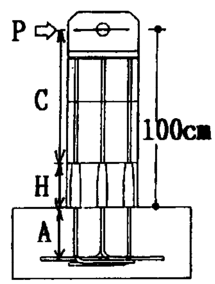

Fig. 10. Definition of the section for analysis.

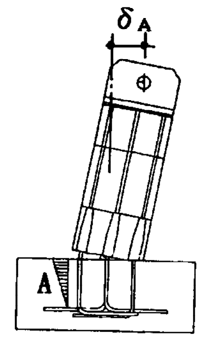

Fig. 11. Displacement due to the rotation at column bottom.

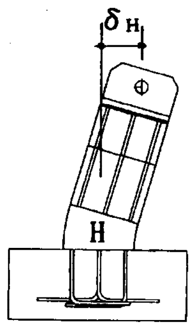

Fig. 12. Displacement due to the flexure in hinge zone.

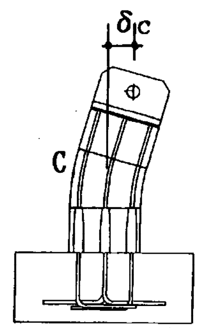

Fig. 13. Displacement due to the flexure except hinge zone.

Analyses vs. Test Results

The load displacement relationships analyzed are compared with the envelope curves of the responses due to the load reversals in Fig. 14 through Fig. 16. The analyses agreed well with the test results, so that the analytical technique may be available to predict the inelastic behavior after flexural yielding on the columns with the tube splices.

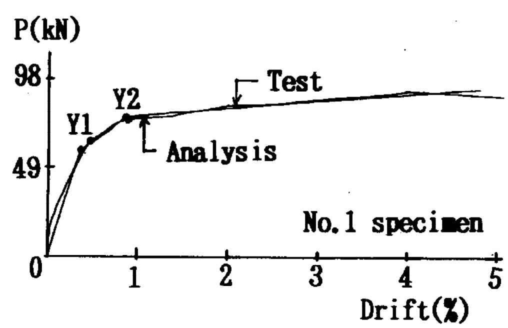

Fig. 14. Load drift relationships analyzed and the envelope curve tested (with steel tube, axial stress: none).

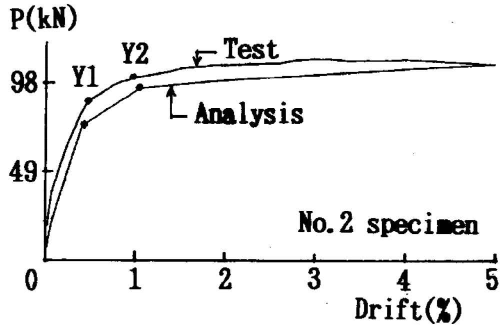

Fig. 15. Load drift relationships analyzed and the envelope curve tested (with steel tube, axial stress: 1.96MPa).

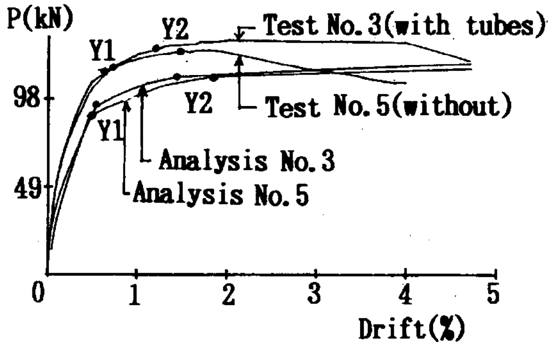

Fig. 16. Load drift relationships analyzed and the envelope curve tested (with steel tube, axial stress: 3.92MPa).

PREDICTION OF POST YIELD OVERSTRENGTH FOR ON COLUMNS WITH TUBE SPLICES

The load increase after flexural yielding generates on the columns with tube splices as observed in Fig. 14 through Fig. 16. The post yield overstrength should be reflected on the design against shear forces of the columns to keep a ductile behavior during a seismic action. The overstrength depends on the lateral displacement of the columns which will be assumed in the structural design. The analytical technique in this paper may be available to predict the overstrength.

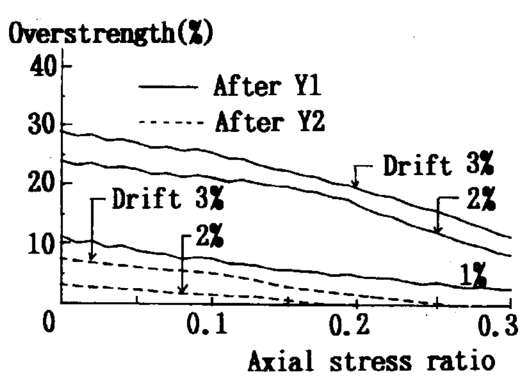

Fig. 17. An example of overstrength predicted.

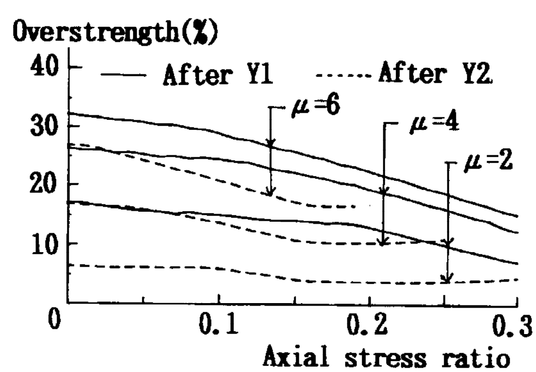

Fig. 18. An example of overstrength predicted.

An example of overstrength prediction is shown in Fig. 17 and Fig. 18. In the prediction, the followings are assumed; percent of the extreme tension and compression reinforcement $\rho = \rho_c = 1.0\%$, the percent of the middle layer of reinforcement $\rho_m = 0.5\%$. The same values of the mechanical properties for steel and concrete was also assumed in the prediction.

The vertical axis of each figure represents the overstrength predicted, and the horizontal axis represents the axial compressive stress of the column. The overstrength corresponding to the drift angles of 1, 2 and 3 percent, respectively, are shown in Fig. 17, and those corresponding to the ductility ratio of 2, 4 and 6, respectively, are shown in Fig. 18. The solid lines in the figures correspond to the overstrength after the yielding of the first layer reinforcement, and the broken lines correspond to that after the yielding of the middle layer reinforcement.

Both figures indicate that the overstrength to be considered on the design shear forces increases as the axial stress decreases. Also the overstrength increases as the design displacement increases.

CONCLUSIONS

The overstrength after flexural yielding for the columns with steel tube splices could be predicted by the inelastic flexural theory using the stress strain model in where the nominal strain hardening was assumed for the steel tube splices. The overstrength ratio essentially depends on the axial stress ratio of the column, so that post yield overstrength should be considered on the design shear forces particularly in the columns subjected to the variable axial load. The increase of flexural strength due to the effect of the multi-layer reinforcement should be primarily considered in the estimation of the design shear forces.

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