Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.IV)

6-7-9

DUCTILE BEHAVIOR OF PARTIALLY PRESTRESSED CONCRETE FLEXURAL MEMBERS WITH CONFINED CONCRETE UNDER SCORES OF HIGH INTENSITY CYCLIC LOADING

Kazuo SUZUKI[1] and Tadashi NAKATSUKA[1]

[1]Department of Architectural Engineering, Osaka University, Suita, Osaka, Japan

SUMMARY

Sufficient ductility is essential for concrete structures to respond safely and satisfactorily to strong seismic attacks. Application of confined concrete to compression zones or whole section of flexural members is exceedingly effective. This paper describes the results of the experimental and analytical investigations on ductile behavior of relatively large sized specimens of partially prestressed concrete beams and columns reinforced by confined concrete which were subjected to scores of high intensity cyclic loading in ultimate plastic stage.

INTRODUCTION

From the point of view of making concrete structures sufficiently safe to seismic load, the establishment of technique and design method to obtain ductile flexural concrete members is urgent need. Though several techniques such as reduction of amount of tensile steel, use of compressive reinforcement and so on improve ductility of flexural concrete members, the application of confined concrete to compression zones of flexural members [1,2,6,7] is considered to be the most effective as shown in Table 1. However, the amount, distribution and configuration of confining reinforcement to ensure adequate ductility are not revealed sufficiently.

The objects of this study is to investigate experimentally the effects of confining reinforcement on ductile behavior and deformation capacities of partially prestressed concrete flexural members subjected to scores of high intensity cyclic loading in ultimate plastic stage, and to investigate analytically ultimate limit curvatures of the sections.

TEST SPECIMENS AND PROCEDURE

Specimens were beam-column assemblies of T-shape as shown in Fig.2. Though there are several arrangement of confining reinforcement in sections as shown in Fig.1, circular spirals and rectangular stirrups for beam specimens and circular spirals for column specimens were employed in the test (see Fig.2). In beam specimens, confining reinforcement were arranged over plastic hinge regions. The experimental variables adopted in the beam specimens were shapes of confining reinforcements , degree of confinement varied by diameters and yield strength of reinforcing bars, and reinforcement index (q_{des}) of such large values of 0.25

and 0.4. Variables in column specimens were the amount of circular spiral confining reinforcement, the coefficient of prestressing in column section, axial force ratio and numbers of cyclic loading.

Test set-up for beam specimens is shown *in Fig.2. Upward and* downward loads were applied to the end part of specimen alternately by two 50tons hydraulic jacks through tension rods. Ten cycles of loading were applied to the beam-end at each target deflection of rotation angle of member (R) such as 1/70, 1/30, 1/20, 1/15 and 1/10. Deflections of the beams at loading point and points at the distance of 0.75D, 1.5D and 3D from the column surface were measured with displacement transducers. To obtain the distribution of curvatures in the hinge region, deformations at both top and bottom sides in the region with four span of 15cm length each were measured with displacement transducers as shown in Fig.3. Test were done in similar way for columns.

Table 1 Advantages and disadvantages of methods to make beams ductile.

Methods		Advantages (○) and disadvantages (▲)
Curtail of amounts of prestressed and/or non-prestressed tensile reinforcement		▲ • Decrement of resistant moment of the section ⟶ Re-design or calculation of the section ○ • Increment of ductility
Enlargement of sizes of section (b,d)		▲ • Change of design stresses in structural frames ⟶ Re-computation of stresses ▲ • Change of resistant moment of sections ⟶ Re-design or calculation of the sections ○ • Increment of ductility
Enhancement of concrete compressive strength		▲ • Necessity of reduction of water cement ratio ⟶ Drop of workability of fresh concrete ▲ • Enhancement of brittleness of concrete ⟶ Small improvement of ductility of concrete flexural members ○ • Increment of ductility
Use of reinforcement to reinforce compression zones of flexural members	Use or addition of compressive reinforcing bars	▲ • Increment of reinforcement index for the action of reverse moment ▲ • Necessity to prevent reinforcing bars from buckling ○ • Increment of ductility
	reinforcement to confine the concrete in compression zone	▲ • Necessity of a new reinforcement to confine the concrete in compression zone, but the amount and region of reinforcement are both small and limited. ○ • Non- or a little increment of resistant moments of sections of sections. ○ • Non- or a little increment of resistant moments of sections ○ • Large increment of ductility ○ • Improvement of deterioration of load carrying capacity under cyclic loads

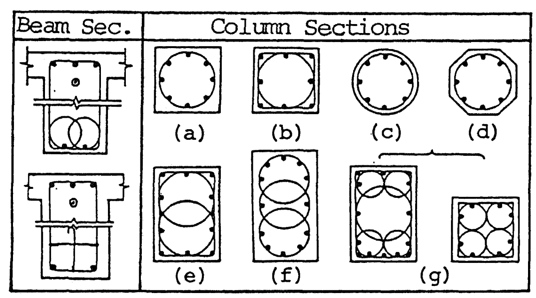
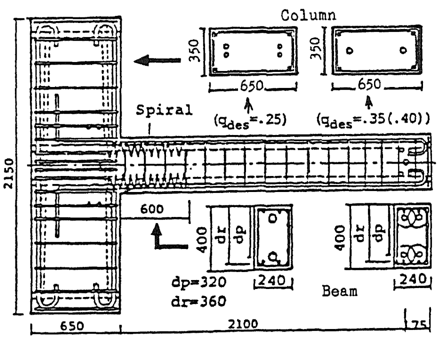


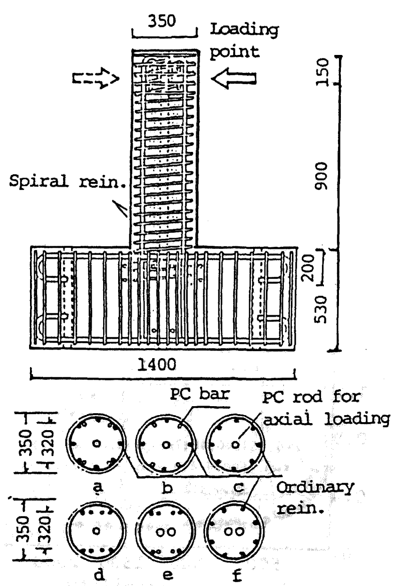
Fig.1 Examples of arrangement of confining reinforcement

TEST RESULTS AND DISCUSSIONS

Displacement Ductility Figures 4 and 5 show results of displacement ductility factors (μ_δ) obtained at the stability limit stages for beam specimens and column ones, respectively (δ_{st}:displacement at the stability limit stage, δ_{ry}:displacement at yielding of tensile reinforcement). In this report "the stability limit" is defined for convenience'sake as the loading stage at which the ratio of load capacity of each Nth cycle (P_N) to that of *each 1st cycle of corresponding loading stage has the* tendency to converge and the value of the ratio (P_{10}/P_1) is less than .85 for beam specimens (.80 for column specimens) as illustrated in Fig.6. The loading stage subsequent to this stability limit stage is termed "the failure stage". Figure 4 indicates that the displacement ductility factor (μ_δ) of beam specimens tends to increase with the increase in amount of confining reinforcement($p_s\sigma_{sy}$) except for the range of large $p_s\sigma_{sy}$ and that the circular spirals are more



(a) Beam specimens



(b) Column specimens

Fig.2 Reinforcement details

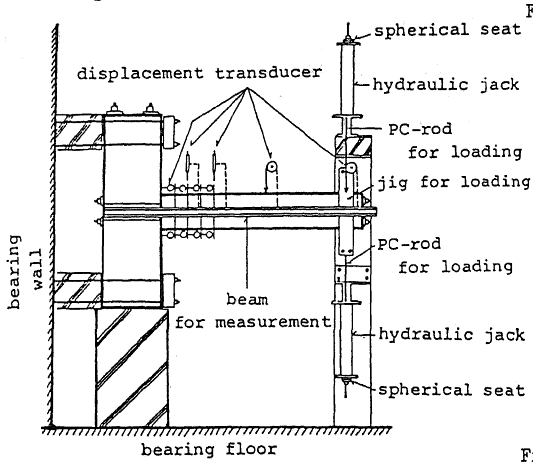


Fig.3 Test set-up

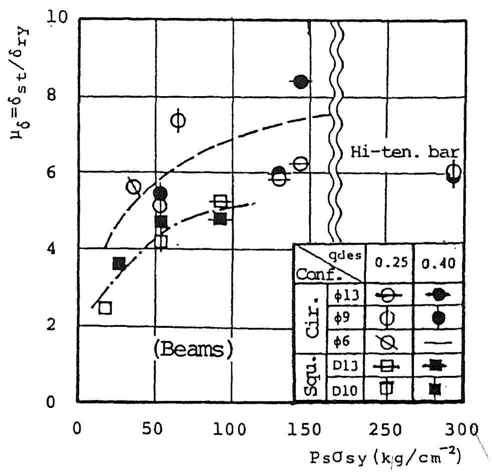


Fig.4 Relation between displacement ductility factor(μ_δ) and amount of confining reinforcement($P_s\sigma_{sy}$)

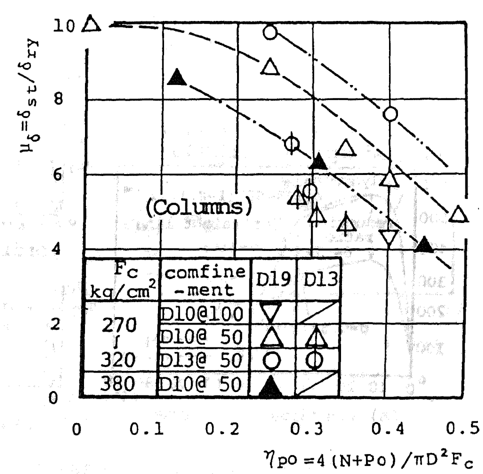


Fig.5 Relation between displacement ductility factor(μ_δ) and axial force ratio(η_{po})

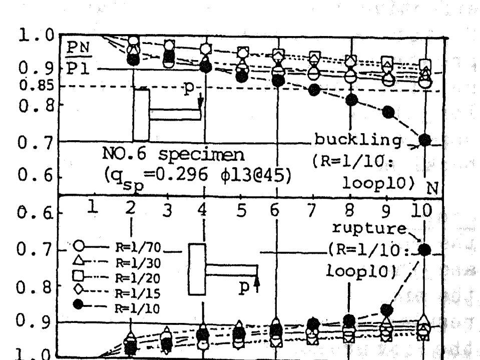


Fig.6 Reduction ratio of load carrying capacity owing to cyclic loads

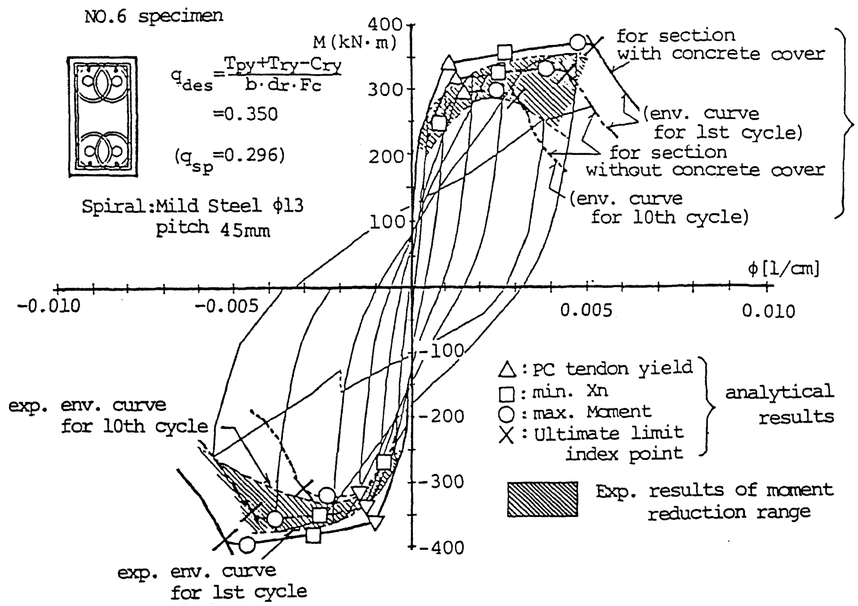
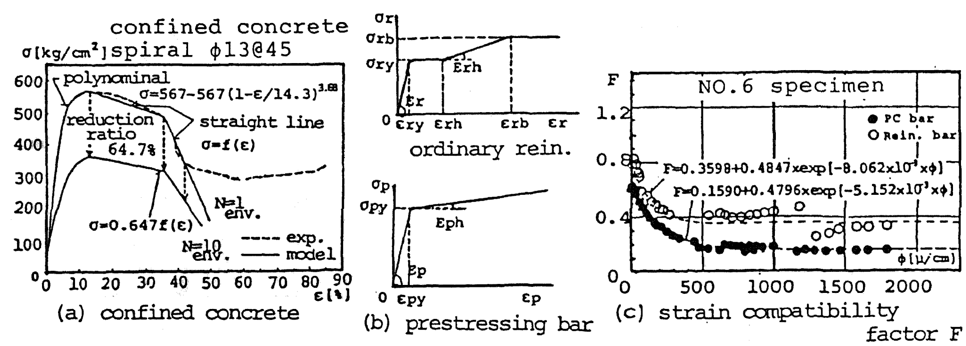


Fig.7 Comparison between experimental envelope curves and analytical moment-curvature relations



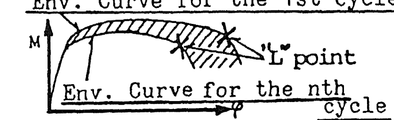
(a) confined concrete (b) prestressing bar (c) strain compatibility factor F

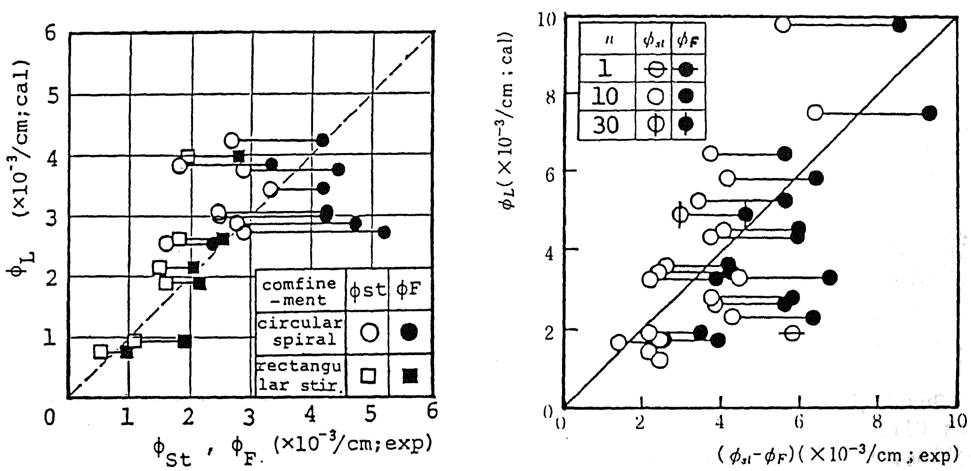
Fig.8 Typical stress-strain curves of concrete and steels, and models of bond deterioration

effective than the rectangular stirrups. From Fig.5 it can be seen that ductility factor μ_δ decreases with the increase in axial force ratios (η_{po}) including prestressing force and increases with the increase in amount of confining reinforcement and reduction of concrete compressive strength. In the case of longitudinal reinforcement of small size (D13), no clear increment of μ_δ is observed despite of increment of confining reinforcement owing to final failure by buckling of longitudinal reinforcement.

Comparison of Moment-Cuvature Relations Figure 7 shows the comparison between the envelope curves of moment-curvature relations obtained through the experiment and the analytical ones calculated from the assumption of monotonical loading. In the analysis the models of stress-strain curve based on the uniaxial repeated test results of confined concrete are used as the stress-strain curves of concrete in the compression zones of beams (see Fig.8(a)). The stress-strain curves of reinforcing bars and prestressing tendons used are shown in Fig.8(b). Furthermore, the bond deterioration of tensile steel bars and prestressing tendons are taken into consideration using strain compatibility factor F (see Fig.8(c)).

Table 2 Features of ultimate limit index point

Definitions	Features
• The point when $[C=T]_{max}$ (i.e maximum value of compressive resultant=tensile one) or ε_s max (max.steel strain) takes place.  • The point when a tensile reinforcement ruptures. • The point when a compressive reinforcement buckles.	The point has clear physical meaning. M and φ at the point is calculated by relatively simple equations. The point is very useful to estimate falling branch of M-φ relations because it always appears in large value of φ after Mmax. The point is considered to be the limit index point of deformation because of a significant decrement of moment capacity after the point. Furthermore the point can be also available to the case of cyclic loads.



(a) Beam specimens (b) Column specimens

Fig.9 Correlations between curvatures at limit index point(ϕ_L) and curvature ranges ($\phi_{St} \sim \phi_F$)

Figure 7 indicates that the experimental envelope curves for the 1st and the 10th cycle of each loading step can be successfully estimated by the analytical ones for which concrete cover of the section is neglected and the stress-strain models of concrete based on the before-mentioned assumptions.

<u>Estimation of Stability Limit</u> For seismic design, analytical estimation of a deflection of stability limit in the load-deflection relation of a flexural member under cyclic loading must be very useful. Though some index points on the moment-curvature relation of flexural members in their ultimate stage are proposed previously, it seems that the "L"points (ultimate limit index points) proposed by the authors are more useful because of features summarized in Table 2. In Fig.9, moreover, ranges of curvatures from the stability limit stage to the final failure one ($\phi_{st} \sim \phi_F$) obtained experimentally are compared with the curvatures at "ultimate limit index point "[3-6] calculated using the degraded stress-strain curves of confined concrete and beam sections without concrete cover under monotonic loading. The good correspondence between the range of curvatures ($\phi_{st} \sim \phi_F$) and the curvature (ϕ_L) suggestes that the ultimate limit index point is an reasonable index to predict the stability limit of a flexural section subjected to high

intensity cyclic loads like seismic attack.

CONCLUSIONS

On the basis of the test results and discussions conclusions are as follows.

(1) Flexual members with sufficient transverse confining reinforcement show spectacular large deformation capacities, even in the case of beams with very large reinforcement indexes and in the case of columns with large axial force under scores of high intensity cyclic loading. Although both the circular and the rectangular confining reinforcements have the effect to increase the ductility of flexural members, the former reinforcement is more effective than the latter (see Figs.4,5 and 7).

(2) Remarkable difference was observed between the envelope curve for the 1st cycle and that of 10th one. This indicates that the mechanical behavior of flexural concrete members against earthquake loading should be evaluated based on relatively large number of cyclic loadings such as ten or more cycles at each loading stage (see Fig.7).

(3) Experimental envelope curves of moment-curvature relationships in the plastic hinge regions of flexural members under cyclic loading can be estimated satisfactorily by the analysis in which concrete cover of sections is neglected and monotonical stress-strain relations of concrete modified from the one under uniaxial repeated loading (see fig.8(a)) is used. The ultimate deformation capacities (final deformations) of sections are closely related to the deformations at the proposed "ultimate limit index point" calculated from the analysis described (see Figs.7,8,9 and Table 2).

REFERENCES

1. Suzuki,K. et al. : Ductile Behavior of Partially Prestressed Concrete Beams with Circular Reinforcement in the Compression Zone: Review of the 38th General Meeting, The Cement Association of Japan, 1984

2. Suzuki,K. et al. : Ductile Behavior and Hysteresis Characteristics of Partially Prestressed Concrete Columns with Circular Spiral Reinforcement: Review of the 39th General Meeting, The Cement Association of Japan, 1985

3. Suzuki,K.: Moment-Curvature Relation and Limit Index Point in Prestressed Concrete and Reinforced Concrete Members: CSCE-ASCE-ACI-CEB International Symposium in Waterioo,Aug.1979

4. Suzuki,K. et al.: Limit Index Point of Prestressed Concrete Flexural Sections in Ultimate State: Technology Reports of the Osaka University, vol.28, April 1978

5. Suzuki,K. et al.: Characteristics of Ultimate Limit Index Point of Partially Prestressed Concrete Flexural Sections: Review of the 36th General Meeting, The Cement Association of Japan, 1982

6. Suzuki.K. et al. : Seismic Resistant Prestressed Multi-Story Buildings - Asseismic Prestressed Concrete Structures with Confined Concrete - ,FIP Symposium ,Israel, Sept.1988

7. Suzuki K. et al. : Ductile Behavior of Partially Prestressed Concrete Beams with Confined Concrete under Scores of High Intensity Cyclic Loading, 10th Australasian Conference on The Mechanics of Structures and Materials, Univ. of Adelaide, 1986