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MOVING AND SPREADING BEAM PLASTIC HINGING ZONES FOR THE DUCTILE BEHAVIOR OF HIGH-STRENGTH REINFORCED CONCRETE BEAM

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ABSTRACT

The purpose of this paper is to propose the reinforcement detail which can make beam plastic hinging zones moved and spreaded from the column face to insure the ductile behavior of high-strength (f'_c=700kg/cm^2) RC beams. The intermediate reinforcement which are vertically anchored by interlinking each intermediate reinforcements is proposed and tested to examine the mechanical performance of proposed details. Main variable is the shape of the intermediate reinforcements. From the test results, the newly proposed intermediate reinforcement details can move and spread the beam plastic hinging zone about 1.0*d* from the column face and can delay the strength decay of the high-strength RC beam. Also the energy dissipation capacity of specimen **IV-1.0D10** which is reinforced by vertically anchored intermediate longitudinal reinforcements about 1.0*d* is 1.6 times as high as the specimen **CM-STAN** which is designed according to ACI 318-89.

KEYWORD

Beam Plastic Hinge; Beam Plastic Hinging Zone; Hysteretic Behavior; Cyclic Load; RC Beam; Intermediate Reinforcement; Energy Dissipation Capacity; Strength Decay; High-Strength Reinforced Concrete

INTRODUCTION

The reinforced concrete frame building designed for code seismic forces will be stressed beyond the elastic limit during a major earthquake. The critical regions in reinforced concrete buildings are usually the beam to column connections. Present codes recommend a "strong column-weak beam" design philosophy to minimize the probability of structural collapse or loss of structural serviceability. Design of reinforced concrete beam-column joints according to the present recommendation of ACI-ASCE Committee 352 results in the development of a beam plastic hinge at the column face. A plastic hinge forming at this location usually cause the stiffness and strength deterioration in the connection.

An alternative approach to solve the beam to column connection problem is to move the beam plastic hinging zone some distance from the column face by adding supplemental intermediate longitudinal reinforcement over a specific length of the beam.

This design concept of moving beam plastic hinge away from the column face can keep the beam section adjacent to the column faces essentially elastic, but relocating the beam plastic hinging zones lead to a larger rotational ductility demand in the beams. Moreover brittle shear failure can be occurred at the relocated plastic hinge because the damage is concentrated with in a narrow area. On the other hand, the reinforcing region of cut-off intermediate longitudinal reinforcements can not be quantified by the irregular bond deterioration of cut-off intermediate longitudinal reinforcements.

The primary purpose of this investigation is to propose the reinforcing details which can make beam plastic hinging zones moved and spreaded from the column face using vertically anchored intermediate longitudinal reinforcements(Fig. 1).

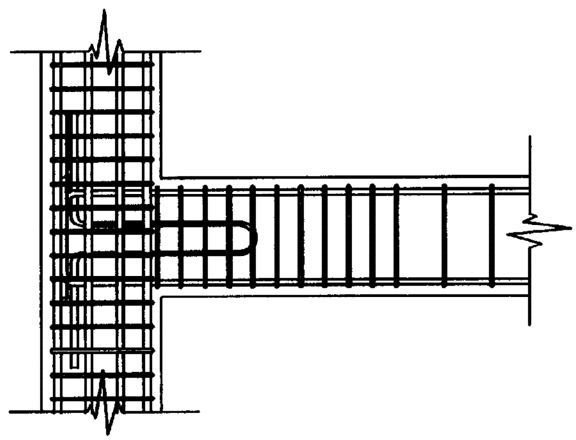


Fig. 1. Vertically anchored intermediate longitudinal reinforcement

EXPERIMENTAL INVESTIGATION

Test Specimens

The cantilever specimen used in this study represents a beam, from midspan to the beam-column joint (Fig. 1). Three cantilever beams with 20×35 cm cross section were tested. Each beam had a shear-span of 1.2 m and a shear-span to effective depth ratio a/d is about 4.0.

The top and bottom reinforcements consist of four No. 5 (16 mm) deformed bars. The flexural reinforcement ratio ρ is 1.06 percent. Intermediate longitudinal reinforcements consisted of four bars placed in two layers at approximately the third points between the tension and compression reinforcement. The ratio of the area of intermediate reinforcement per layer to main tension reinforcement is 0.18. The specimen named **CM-STAN** is reinforced according to the present ACI 318-89, the specimen named **IC-1.5D10** is reinforced by adding the supplemental cut-off intermediate bars, and the specimen named **IV-1.0D10** is reinforced by adding the vertically anchored intermediate bars. A summary of beam and reinforcement properties is presented in Table 1.

Table 1. Specimen list

Specimen	Width b (cm)	Depth D (cm)	Flexural reinforcement A_s	Flexural reinforcement ratio ρ (%)	Intermediate reinforcement A_i	Intermediate reinforcement: hook	length	$\frac{A_i}{A_s}$	Stirrup
CM-STAN	20	35	4-D16	1.06	-	-	-	-	φ6-@65
IC-1.5D10	20	35	4-D16	1.06	2-D10	Cut-off	1.5*d*	0.18	φ6-@55
IV-1.0D10	20	35	4-D16	1.06	2-D10	Vertical	1.0*d*	0.18	φ6-@55

- f_y = Yield strength of flexural reinforcement (4000 kg/cm^2)
- f_{yi} = Yield strength of intermediate reinforcement (4000 kg/cm^2)
- f_{ys} = Yield strength of shear reinforcement (4000 kg/cm^2)
- f'_c = Compressive strength (700 kg/cm^2)
- Dimension of column section : 40 x 40 cm

The total shear capacity of each beam V_n is satisfactory to insure a flexural failure under monotonic loading. The nominal shear stresses are computed in accordance with the provisions of ACI 318-89. The first stirrup is placed on one inch from the column face.

Material

Concrete, with 13 mm nominal maximum size aggregate, is obtained from local ready-mixed plant. Compression cylinders measuring 10×15 cm are prepared for each beam. Concrete strengths are listed in Table 3.

The flexural reinforcements for the beams consist of No. 4 ASTM A615 Grade 60 deformed bars. The longitudinal reinforcements for the column-stub consist of No. 5 deformed bars. Column ties and Hoops are made of No 1. (6 mm) deformed bars.

Table 2. Tensile test of the reinforcements

Type	Elastic modulus (t/cm^2)	Yield strength (t/cm^2)	Yield strain (x 10^{-6})	Tensile strength (t/cm^2)	Elongation (%)
HD19	2117	4.12	1946	6.70	21.0
HD16	1987	4.09	2058	6.45	19.8
HD10	1928	3.87	2007	5.75	21.4
HD6	1807	4.38	2420	5.70	15.3

Table 3. Compressive test of the concrete

Type	Compressive strength (kg/cm^2): 3 day	7 day	14 day	28 day	Height (cm)	Area (cm^2)	Elastic modulus (t/cm^2)	Slump (cm)
f'_c = 700 kg/cm^2	590	610	620	710	20.2	78.6	217	21.4

Instrumentation

Electrical resistance strain gages are bonded to the longitudinal reinforcement and to stirrups placed within a distance equal to the beam depth from the column face. Linear variable displacement transformers (LVDTs) are used to measure the beam deflection at a distance of 1.2 m from the column face and the shear deformation over three regions, 0.5*d* adjacent to the column face, 0.5*d*~1.0*d* and 1.0*d*~2.0*d*.

Test procedure

During testing, the specimens are held vertically in a steel frame with pin supports near the end of the beam and column (Fig. 2). The actuator is attached to a steel frame prestressed to the structural floor of the laboratory. The constant axial force corresponding to $0.2f'_c$ (192 ton) is applied at the column prior to the hysteretic beam loading. Load is applied using a 50 ton servo-hydraulic actuator under displacement control. The beam is, first, loaded upward (positive bending) slightly beyond the yield strain of the longitudinal reinforcement. The yield displacement, δ_y, is set when the flexural reinforcement reaches its yield strain. The direction of loading is then reversed. The loading history scheduled is shown in Fig. 3. Test is proceeded until the maximum cycle load, P_n, drop 75% of its yield load P_y, which is defined as the collapse load.

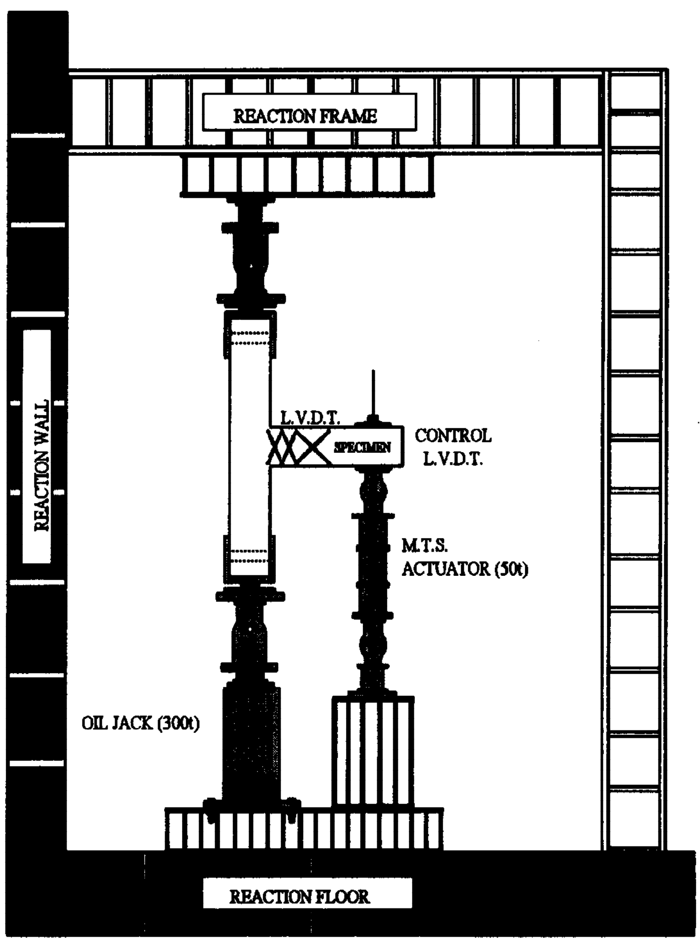


Fig. 2. Schematic drawing of the testing frame

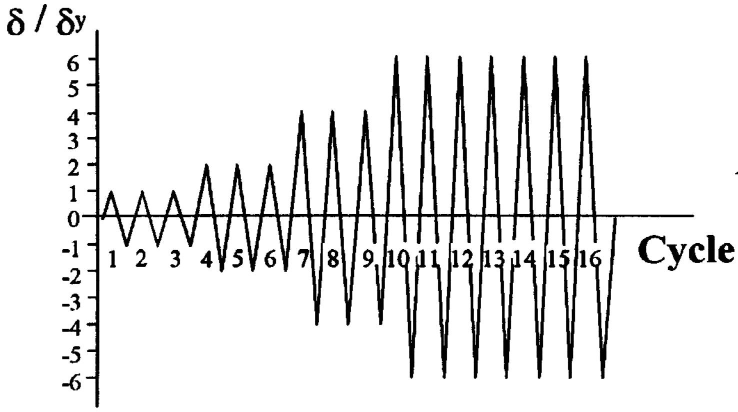
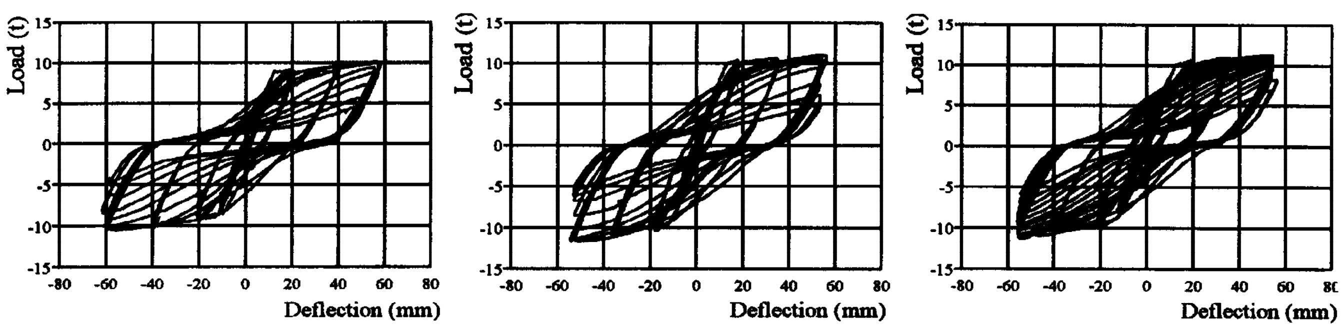


Fig. 3. Loading history

Test results

Load-Deflection Curve Fig. 4 shows the load-deflection curve of each specimen. Each specimen undergoes the same loading history. The specimen **CM-STAN** showed the drastic strength deterioration after 12 cycle ($6\delta_y$) and was finally collapsed at 14 cycle due to the shear fracture of the confined core concrete. The specimen **IC-1.5D10** also showed the strength deterioration after 13 cycle but less serious than the specimen **CM-STAN** and was finally collapsed at 16 cycle as the intermediate longitudinal reinforcements buckled as shown in Fig. 5.

On the other hand, the specimen **IV-1.0D10** showed sustained load-carrying capacity up to the final failure cycles. It is originated by the fact that the intermediate longitudinal reinforcement was to inhibit opening of crack in the beam hinging zone, promoting a more uniform distribution of cracking of concrete and preventing localized failure. Therefore, the beam plastic hinging zone of the specimen **IV-1.0D10** is moved and spreaded by reinforcing the vertically anchored intermediate longitudinal reinforcements over the 1.0*d* region from the column face.



(a) Beam, CM-STAN (b) Beam, IC-1.5D10 (c) Beam, IV-1.0D10

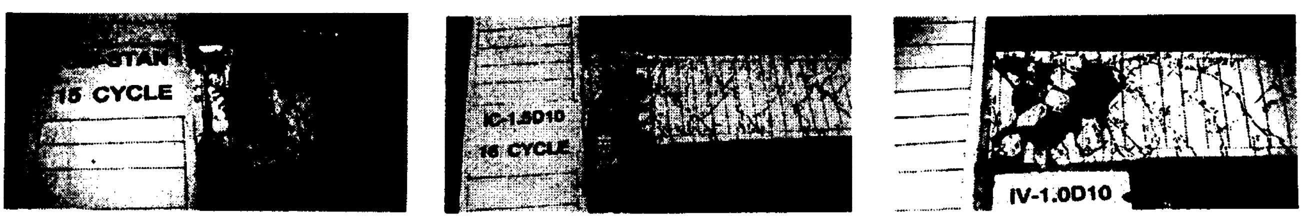
Fig. 4 Load-deflection curve

Cracking Pattern at Failure and shear deformation of plastic hinge Fig. 5 shows the cracking pattern of each specimen at failure and Fig. 6 shows the relationships between the shear deformation and each load cycle. The shear deformation of three regions adjacent to column face is measured by the six LVDTs attached to the beam plastic hinging zone.

The failure of specimen **CM-STAN** was initiated by the spalling of the cover concrete and finally collapsed by the diagonal shear fracture of the core concrete at 0.5d from the column face. And the magnitude of shear deformation at D_1(0.5d) region is much greater than that of D_2 (0.5d~1.0d) and D_3 (1.0d~2.0d) as shown in Fig. 6.

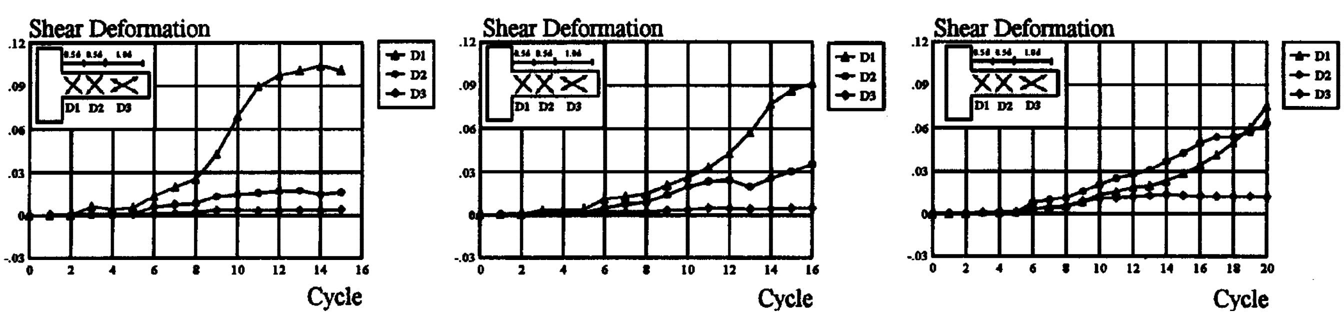
Also the spalling of the cover concrete at 0.5d caused the failure of specimen **IC-1.5D10.** However the severe fracture of the core concrete was not observed up to 13 cycle, and finally it was collapsed by the buckling of cut-off intermediate longitudinal reinforcements. Such delayed failure mode can be observed in the relationships between shear deformation and load cycle at each region, but its relative magnitude of shear deformation at each region is similar to that of specimen **CM-STAN.**

On the other hand, the failure of specimen **IV-1.0D10** was initiated at 1.0d from the column face and propagated to the column face. Finally dispersed cracking pattern was found. Also the magnitude of shear deformation at D_1 (0.5d) region is similar to that of D_2(0.5d~1.0d). Moreover shear deformation of specimen **IV-1.0D10** at D_1 (0.5d) region is reduced compared with that of specimen **CM-STAN** and **IC-1.5D10** about 60 percents. It is originally based on the fact that shear resistance capacity of specimen **IV-1.0D10** is increased by the additional confining effects of core concrete and dowel action provided by the intermediate longitudinal reinforcement. The increased shear resistance capacity made the beam plastic hinging zone spreaded, lead to decreased rotational ductility demand of beam.



(a) Specimen CM-STAN (b) Specimen IC-1.5D10 (c) Specimen IV-1.0D10

Fig. 5. The cracking pattern at failure



(a) Specimen CM-STAN (b) Specimen IC-1.5D10 (c) Specimen IV-1.0D10

Fig. 6. The shear deformation versus load cycle

Energy Dissipation Capacity The ability of the member to dissipate energy is perhaps the most important aspects of structural performance under seismic loading. The "energy dissipated" is taken as the area enclosed by the load-deflection curve. Only cycles for which the peak load P_n is greater than 75 percent of the initial yield load P_y are considered in the computation of the dissipated energy E. Energy dissipation values based on this criterion are presented in Table 4.

The energy dissipation capacity of specimen **IV-1.0D10**, which is reinforced by vertically anchored intermediate longitudinal reinforcements about 1.0*d* is 1.6 times as high as that of the specimen **CM-STAN**, which is designed according to ACI 318-89. Also the energy dissipation capacity of specimen **IV-1.0D10** is 1.4 times as high as that of the specimen **IC-1.5D10**, which is additionally reinforced by cut-off intermediate longitudinal reinforcements.

Table 4. Test results

Specimen	Yield load P_y(t)		Maximum Load P_m(t)		Yield displacement Δ_y(cm)		Maximum displacement Δ_m(cm)		Cycle (n)			Dissipated energy (t·cm)
									P_n>0.75P_y		Total	
	+	−	+	−	+	−	+	−	+	−		
CM-STAN	7.33	8.30	10.07	10.45	0.99	1.09	5.80	6.14	14	13	15	565.5
IC-1.5D10	7.31	6.94	10.84	11.62	0.91	0.87	5.60	5.28	15	15	16	677.4
IV-1.0D10	8.37	7.95	11.25	10.92	0.95	1.21	5.62	5.59	20	19	20	898.8

CONCLUSIONS

Three reinforced concrete cantilever beam specimens were constructed and tested to study the effects of the vertically anchored intermediate longitudinal reinforcement in moving and spreading beam plastic hinging zone away from the column face. Based on the results of these tests, the following conclusions can be drawn.

The proposed intermediate longitudinal reinforcement details can make the beam plastic hinging zone moved and spreaded about 1.0*d* from the column face.

The vertically anchored intermediate longitudinal reinforcements can delay the strength decay of high-strength R/C beams.

The energy dissipation capacity of specimen **IV-1.0D10** is the most excellent; the energy dissipation capacity of specimen **IV-1.0D10**, which is reinforced by adding vertically anchored intermediate longitudinal reinforcements about 1.0*d* is 1.6 times as high as that of the specimen **CM-STAN**, which is designed according to ACI 318-89.

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