Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Test of reinforced concrete beams with lap splices at hinge region

Manabu Yoshimura & Kozo Tsumura
Tokyo Metropolitan University, Japan

ABSTRACT: Seismic Behavior of reinforced concrete beams with lap splices at hinge region was experimentally investigated. Test variables were shapes and length of lap splices. Major findings from the tests were as follows; 1) In case that lapped bars did not have a hook at the tip, the pullout of these bars from the beam could not be avoided even if the length of splices was as much as $40d$ (d:bar diameter), but 2) In case that lapped bars had a hook, favorable seismic behavior similar to the specimen without splices was obtained if the length of splices was $20d$.

1 INTRODUCTION

In most reinforced concrete construction, longitudinal reinforcing bars are spliced at the location of members far from the hinge region where large forces may arise during earthquakes. However, if the placement of splices at such region may become possible, it will increase to a great extent the versatility of design and construction practice of reinforced concrete. And it will be especially valuable when precast concrete members such as shown in Fig.1 are used. Considering there have been few past researches along this objective (Tanaka 1989), an attempt is made in this paper to investigate the seismic performance of reinforced concrete beams with lap splices at the hinge region experimentally.

2 OUTLINE OF TEST

Specimens, total nine in number, are outlined in Table.1, and their reinforcement details and shapes of lap splices are shown in Figs.2 and 3. Splices were placed for bottom bars but not for top bars, simulating the precast concrete beam as illustrated in Fig.1. However, note that all portions of the specimens were rendered cast-in-place concrete for the sake of simplicity of construction. Longitudinal and lateral reinforcement was the same for all specimens. Test variables were shapes and length of lap splices. Three sorts of a shape were considered depending on the presence and detail of a hook; no hook (LS-2~LS-5), 90° hook (LS-6~ LS-7) and 180° hook (LS-8~ LS-9). In addition to these, a specimen without splices (LS-1) was fabricated as a standard one.

Mechanical properties of materials used for the specimens are shown in Table 2. Concrete cylinders were tested on the first day and the last day of the testing period. Compressive strength of concrete was $21.5MPa$ and yield strength of reinforcement was 350 MPa and $379MPa$ respectively for $D16$ and $D6$,where the number after D denoted a bar diameter in mm.

The specimens were tested as a simple beam by $1000KN$-capacity oil jack in both senses (Fig.4). To compare hysteretic behavior, all specimens were subjected to the same loading history. The sequence of loading was as follows; one cycle at $R = \pm 1/1000$ and $\pm 1/400$, two cycles at $R = \pm 1/200$, $\pm 1/100$ and $\pm 1/50$, and half cycle in the positive direction upto failur, where R was an average of left and right beam deflections divided by beam clear span.

In addition to beam deflections, strains of longitudinal and lateral reinforcement were measured by strain gages.

3 TEST RESULTS

The state of the specimens observed after the tests is illustrated in Fig.5 for LS-2 and LS-7. They were

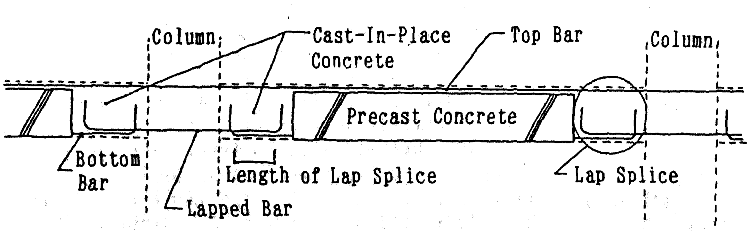
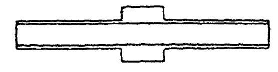
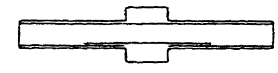
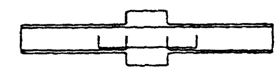
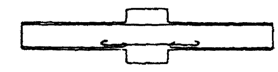


Fig.1 Example of Locating Lap Splices at Hinge Region

Table 1 Outline of Specimens

Specimen	Name	Shape of LS *	Length of LS
	LS-1	-	-
	LS-2 LS-3 LS-4 LS-5	No Hook	40d 30d 20d 10d
	LS-6 LS-7	90°Hook	20d 10d
	LS-8 LS-9	180°Hook	20d 10d

* Note: LS=Lap Splice

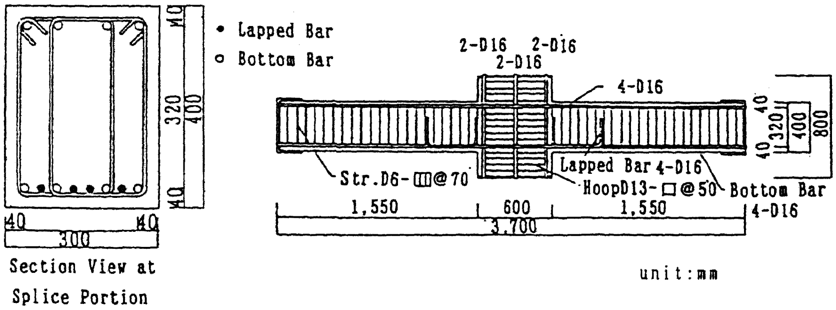
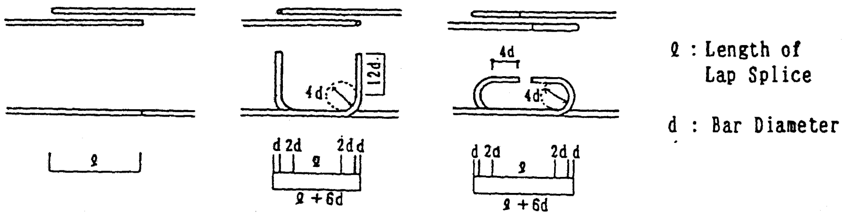


Fig.2 Reinforcement Details of Specimen (LS-6)



a) No Hook b) 90°Hook c) 180°Hook

Fig.3 Shapes of Lap Splices

selected respectively as representatives of 'No Hook' specimens (LS-2 ~ LS-5) and 'Hook' specimens (LS-6~LS-9). Section views near the lapped bar in the figure were depicted after cutting the specimens along these sections. Envelopes of load-deflection relations are compared in Fig.6 for the positive loading (bottom in tension) separately for the No Hook specimens and Hook specimens, in which the load was defined as beam shear.

Test results are outlined below.

1.The standard specimen (LS-1) exhibited stable hysteresis up to $R=1/25$ without any strength drop. Such favorable behavior was due to extensive flexural yielding of the bottom bars.

2.All No Hook specimens (LS-2~LS-5) failed with drastic strength drop at a deflection level less than $R=1/50$. Such sudden failure was due to the pullout of the lapped bars from the beam. As a result of this, a $10mm$ gap between the tip of the lapped bar and surrounding concrete and bond splitting cracks at the splice portion were observed for LS-2 (Fig.5.a)). Similar observations, although not shown here, were made for all No Hook specimens. The length of splices (ℓ) vitally affected the hysteresis of these specimens. A deflection level(Ru), at which sudden strength drop was observed, tended to be larger as ℓ became longer; Ru = 1/400,1/200,1/100 and 1/50, respectively for LS-5, LS-4, LS-3 and LS-2. Note that for LS-2 and

Table 2. Mechanical Properties of Materials

a) Concrete

Fc (MPa)	Ec (MPa)	ν
21.5	2.01×10^4	0.203

Note)
Fc:Compressive strength
Ec:Young's modulus
ν :Poisson's ratio

b) Reinforcement

Size	f_y (MPa)	Es (MPa)	f_m (MPa)	ε_u (%)
D16	350	1.86×10^5	500	17.4
D 6	379	2.19×10^5	534	16.0

Note) f_y:Yield strength
Es:Young's modulus
f_m:Maximum strength
ε_u:Strain at fracture

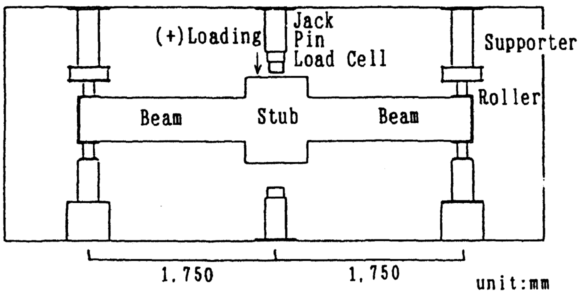


Fig.4 Loading Setup

LS-3, flexural yielding of the lapped bars was observed prior to the pullout of these bars, while it was not detected throughout the tests for LS-4 and LS-5.

3.The Hook specimens (LS-6~LS-9) showed considerably better hysteretic behavior than the No Hook specimens. The hysteresis of LS-6 (90° hook) and LS-8 (180° hook) both having $\ell = 20d$, was close to that of LS-1, except that strength decay of some extent was observed for these specimens during the load reversal of $R = 1/50$. LS-7(90° hook) and LS-9(180° hook) both having $\ell = 10d$, showed considerable strength decay during the load reversal of $R = 1/100$, but the hysteresis hereafter was stable. The pullout of the lapped bars was not observed for the Hook specimens; instead these specimens failed due to the bond splitting along the entire length of the lapped bars including a bent portion. The spalling of cover concrete along the lapped bars related to such failure was observed for all Hook specimens, although the extent of it was less severe for LS-6 and LS-8 than for LS-7 and LS-9 (Fig.5.b)).

4 BOND BEHAVIOR OF LAPPED BARS FOR NO HOOK SPECIMENS

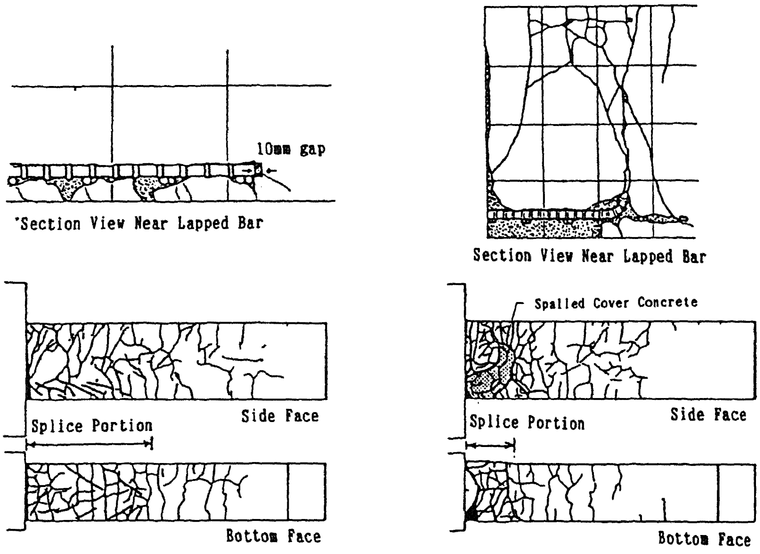
Bond stress between the lapped bar and concrete was evaluated for the No Hook specimens, based on the *strain measurement made on several points of the* lapped bar. Bar stress was determined using the results of the material tests. Bond stress was computed from bar stresses at adjacent two points.

The bond stress of the lapped bar is shown in Fig.7 for LS-2. The plots were made for each $10d$ length of the lapped bar for different deflection levels. The region near the beam end (critical section) took a higher bond stress in early stages of loading, but as the test proceeded,the region of high bond stress was moving toward the tip of the lapped bar. This was because the yielding of the lapped bar propagated inside the beam.

Maximum values of bond stress for each $10d$ length of the lapped bar throughout the tests are plotted in Fig.8 for all No Hook specimens. Since pullout failure of the lapped bars was observed for all these specimens, the values shown in this figure could be considered as bond strength for each region. The bond strength showed the highest value ($4MPa$) at the $30d \sim 40d$ region of LS-2, which was the farthest from the beam end, and tended to decrease as the region was approaching the beam end. Such trend of bond strength was believed to be due to that as the region was approaching the beam end, the bond resistance might decrease because of the effect of flexural yielding.

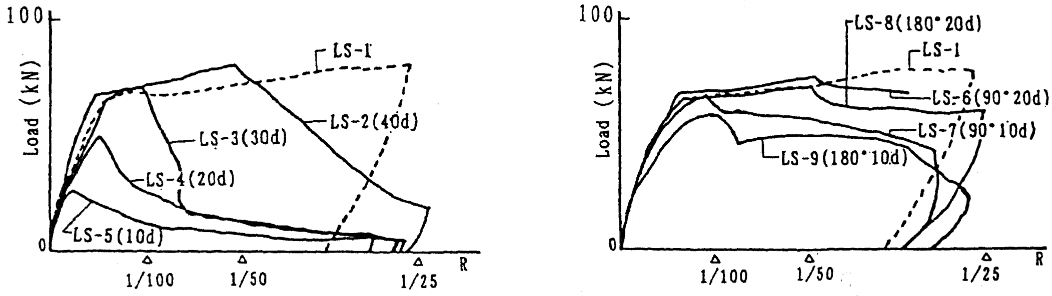
Average bond stress computed over the entire length of the lapped bar for different deflection levels is shown in Fig.9 for all No Hook specimens. Maximum values of average bond stress of LS-5 and LS-4 lay within $2.6 \sim 3.1MPa$ ($0.12 \sim 0.14Fc$). Since both specimens failed due to the pullout before yielding, these values could be considered as average bond strength. Note that LS-2 showed relatively smaller average bond stress because of the effect of yielding.

Maximum values of avarage bond stress are shown in Fig.10 with average bond strength (τ) computed by the following equation developed by Orangun et al.(Orangun 1977).



a) LS-2 (No Hook, ℓ=40d)　　b) LS-7 (90°Hook, ℓ=10d)

Fig.5 State of Specimens after Tests



a) No Hook Specimens　　b) Hook Specimens

Fig.6 Comparison of Envelopes of Load-Deflection(R) Relations

$$\tau = 0.0830 \cdot \sqrt{Fc} \cdot \left(1.2 + 3\frac{c}{d} + 50\frac{d}{\ell} + \frac{p_w \cdot f_y \cdot b}{3.43 \cdot n \cdot d}\right) \quad MPa \qquad (1)$$

where,

$p_w \cdot f_y \cdot b/(3.43 \cdot n \cdot d) \leq 3$

Fc : compressive strength of concrete (MPa)

c : half the clear spacing between two bars (see Fig.2)

d : bar diameter

ℓ : length of lap splice

p_w : ratio of lateral reinforcement

f_y : yield strength of lateral reinforcement (MPa)

b : beam width

n : number of lap splices in the section

In Fig.10, average bond stress computed assuming the lapped bar yields at the beam end and short-term allowable bond stress prescribed in the AIJ(Architectural Institute of Japan) Standard for Structural Calculation of RC Structure are also shown as a reference. The observed average bond strength of LS-5 and LS-4 was considerably smaller than the computed values. This result seems to suggest that the Orangun's equation, which was derived from the tests mostly conducted under constant moment distribution, may give unconservative estimation of average bond strength if applied to lap splices placed at the hinge region.

The maximum values of average bond stress were smaller than the AIJ short-term allowable bond stress for all specimens.

5 ANCHORAGE BEHAVIOR OF LAPPED BARS FOR HOOK SPECIMENS

Measured strains of the lapped bar are shown in Fig.11 for LS-6. The plots were made for several

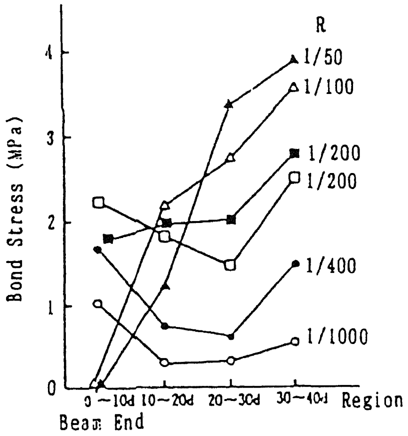


Fig.7 Bond Stress of Lapped Bar (LS-2)

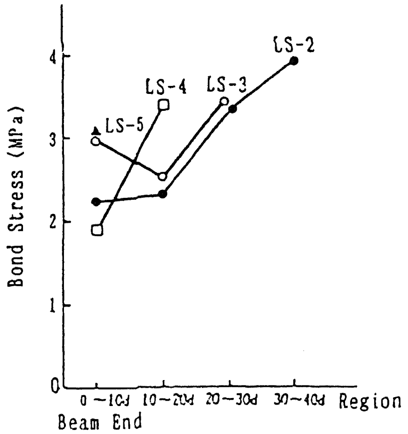


Fig.8 Maximum Bond Stress of Lapped Bar

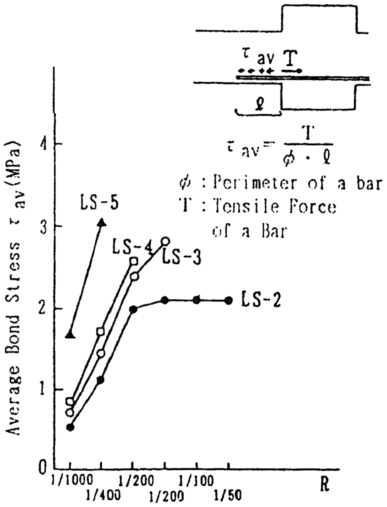


Fig.9 Average Bond Stress of Lapped Bar

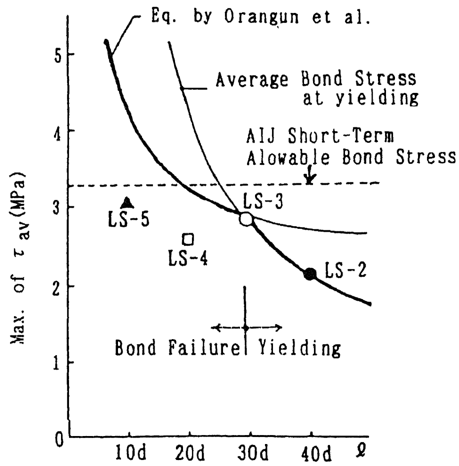


Fig.10 Comparison of Observed and Computed Average Bond Strength of Lapped Bar

points of the lapped bar for different deflection levels. The strains tended to be small as the location was distant from the beam end and the strains near the hook ($21d$ distant from the beam end) did not reach a yielding level even at $R=1/50$. The yielding of the lapped bar near the hook was not observed for any Hook specimen. Maximum tensile force induced in the hook was evaluated from the maximum strain observed near the hook and listed in Table 3.

In order to estimate maximum tensile force of a 90° hook, the model(AIJ 1990) proposed to calculate maximum pullout force of an anchoraged bar in an exterior beam-column joint was applied. In this model the maximum tensile force of a hook (P) was assumed to be determined by failure of the concrete compression field(Fig.12,Eq.(2)). Computed values are also listed in Table 3.

$$P = w \cdot d \cdot f_{bear} \cdot \sin\theta \qquad (2)$$

where,

w : width of concrete compression field
d : bar diameter
f_{bear} : compressive strength of concrete compression field

See the reference(AIJ 1990) for the details.

The computed values were larger to some extent than the observed values for LS-6 and LS-7. Such difference was probably due to that concrete damage in the hinge region was severer than that in the beam-column joint.

6 CONCLUSIONS

From the tests of beams with a variety of lap splices at the hinge region, the following conclusions were drawn.

1.All No Hook specimens failed due to the pullout of the lapped bars and showed drastic strength drop

Table 3 Maximum Tensile Force of Hook

Specimen	Computed (kN)	Observed (kN)
LS－6 (90° 20d)	83.2	49.2
LS－7 (90° 10d)	64.2	49.2
LS－8 (180° 20d)	－	56.2
LS－9 (180° 10d)	－	45.1

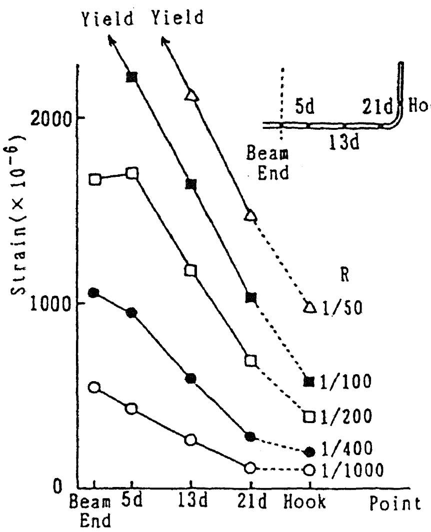


Fig. 11 Strain of Lapped Bar(LS-6)

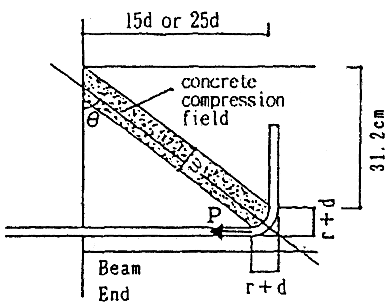


Fig. 12 Concrete Compression Field Formed by 90° Hook

at that moment. Although deflection levels at failure were larger as the length of splice (ℓ) bacame larger, even LS-2 having $\ell=40d$ could not avoid such failure. The observed average bond strength was smaller than the computed value by the existing equation.

2.All Hook specimens showed better deformability than the No Hook specimens. LS-7 (90° hook) and LS-9 (180° hook) both having $\ell = 20d$, showed favorable deformability close to the specimen without splices.

ACKNOWLEDGMENT

The authors wish to express their gratitude to Dr. Y.Kurose,Shimizu Construction Company,Japan, for his invaluable advice in determining the test program.

REFERENCES

AIJ. 1990. Design guidelines for earthquake resistant reinforced concrete buildings based on ultimate strength concept:164-166.(in Japanese)

Orangun,C.O. Jirsa,J.O. et al. 1977. A reevaluation of test data on development length and splices. ACI Journal. Vol.74: 114-122.

Tanaka,R. et al. 1989. Experimental study on the locations of lap splices in the longitudinal bars of reinforced concrete beams. Journal of structural and construction engineering. Transactions of AIJ. No.396: 69-80.(in Japanese)