THE BENEFICIAL EFFECTS OF SHEAR REINFORCEMENT ON THE SEISMIC BEHAVIOUR OF FLAT PLATE STRUCTURES

by

Neil M. Hawkins[1], Denis Mitchell[1] and Samir N. Hanna[1]

[1]Professor, Assistant Professor and Graduate Student, respectively, Department of Civil Engineering, University of Washington, Seattle, Washington.

ABSTRACT

This paper reports one phase of a research program at the University of Washington on the seismic response of flat plate to column connections. In particular the behaviour of connections containing shear reinforcement in the slabs is discussed. The paper compares different design approaches, outlines the benefits of placing shear reinforcement in the slab and gives guidelines as to the design and proper detailing necessary for that shear reinforcement. The effects of concentration of the flexural reinforcement in the immediate column region are also examined. Ductility ratios, energy absorption, energy dissipation, and degeneration of stiffness characteristics of the specimens are reported together with the lateral loads for first yielding and maximum capacity of the specimens.

INTRODUCTION

Primarily because of recent collapses during earthquakes and the known lack of ductility and energy absorption of slab-column connections[1], any flat plate framing is generally neglected for evaluations of the seismic resistance of concrete structures[2]. Many structures are, however, built with such framing. Usually flat plate framing, by itself, exists only in the upper floors of structures whereas at lower levels this type of framing is used in conjunction with a primary, moment resistant, ductile frame. During an earthquake, there is a possibility of the slab-column connections failing and contributing significantly to the damage of the structure. A recent test program conducted at the University of Washington indicates that major damage can readily be avoided by the provision of carefully detailed shear reinforcement in the slab. In this paper results are reported of tests on four slab-column specimens containing shear reinforcement. The results of those tests are compared with the results of tests on similar specimens without shear reinforcement[3].

SIMULATION OF SLAB-COLUMN CONNECTION

Specimen dimensions were chosen so as to permit a realistic examination of the behaviour of slab-column connections under constant dead load coupled with reversed cyclic lateral loads[3]. The test specimens represented to approximately full scale the portion of a flat plate structure extending from an interior column out to the probable region of contraflexure for a moderate earthquake loading and between points of contraflexure in the column. The prototype flat plate structure had a 6 in. (3.96cm.) thick slab with 20 ft. (6.1 m.) square panels. The experimental slab measured 13 ft. (3.96 m.) in the direction of seismic motion, was 7 ft. (2.13 m.) wide and 6 in. (15.24 cm.) thick. The column was 12 in. (30.48 cm.) square, extended 4 ft. (1.22 m.) above and below the slab and was prestressed to simulate axial compression. The test set-up is shown in Fig. 1 and idealization of the specimen from the prototype structure is discussed in greater detail in Reference 3.

The testing apparatus contained two separate jacking systems allowing independent variations of the "gravity" and "lateral" loads shown in Fig. 1. Jacks for the gravity load system were connected to a common pumping source and backed by an accumulator that allowed them to float as the slab rotated. Jacks for the lateral load system were of the push-pull type. They acted in opposite directions for opposite ends of the slab and were connected to a second common pumping source.

EXPERIMENTAL PROGRAM

The purpose of the experimental program was to determine the following:

(1) The size, spacing and length of shear reinforcement necessary in the slab to provide a ductile behaviour with large energy absorption and dissipation characteristics.

(2) Proper detailing of stirrup shapes and anchorages in order to ensure ductility and adequate performance of slab-column connections.

(3) The residual vertical load capacity of connections that have undergone reversed cyclic loadings comparable to those likely in a major earthquake.

and (4) The effects of concentrating the flexural reinforcement around the column, within a distance of one and one half times the slab thickness, h, either side of the column.

Details of the specimens tested are given in Table 1. For Specimens SS2 and SS5 the amount and distribution of the slab reinforcement was the same as that for specimen S2 without shear reinforcement reported in Reference 3. Similarly specimens SS3 and SS4 were identical to specimen S5 and specimen SS1 was identical to specimens S1 and S4. For the tests reported here the main variables were the amount and distribution of the flexural reinforcement and the size, spacing, detailing and length of the closed stirrup reinforcement.

The size and spacing of the stirrups were determined using ACI 318-71 Code provisions[2] for design of shear reinforcement and for the transfer of moments to slab-column connections. A shear capacity was provided equal to 1.2 times the capacity for development of the flexural reinforcement across the full width of the slab on a line passing through the column face.

Each test specimen was carefully instrumented to provide detailed data on its behaviour throughout its entire loading history. The applied gravity loads and the lateral loads were monitored by means of load cells positioned at the loading points indicated on Fig. 1. A combination of linear potentiometers, dial gauges and deflection scales were used to determine deflections and rotations at selected points on the specimen's surface. The instrumentation was designed to permit accurate calculations of slab tip deflections, column rotations and deflections, slab rotations with respect to the column, twists of the portion of the slab adjacent to the column, and the deflected profile of the centerline of the slab. Electrical resistance strain gauges were used to determine strains at selected locations on the slab reinforcement, the stirrup legs, and the concrete. The strain measurements permitted determination of the load for yielding of every bar passing through the column and the spread of yielding across the width of the test specimen. A computer system was developed to provide on-line control of the experiment and to acquire, reduce and store the experimental data in real time as each test proceeded.

DISCUSSION OF TEST RESULTS

(1) The Benefits of Shear Reinforcement

In Figs. 2, 3 and 4 west lateral load versus specimen edge deflection envelopes are compared for specimens with and wihtout shear reinforcement and having the same amount of flexural reinforcement. Specimen edge deflections were measured at the centerline of the specimen on the west lateral load line. Positive values of load and deflection correspond to a downward loading on the west lateral load line. Cycling was always commenced with downward loading on the west lateral load line.

The benefits of shear reinforcement are readily apparent from Fig. 2, 3 and 4. The ductility, energy absorption and strengths characteristics of each connection are markedly improved. Further, the degree of improvement increases as the reinforcement ratio in the slab decreases. Seismic response parameters were determined, as shown in Fig. 5, from the moment transferred to the column versus slab rotation data for cycles at "first yield" and "ultimate" load. Those parameters are listed in Table 2.

Values are given for the stiffness at first yielding, λ_y , the ratio of the stiffness at ultimate to the stiffness at first yield ($\frac{\lambda_u}{\lambda_y}$), the ductility ratio at ultimate load (μ_u) and the damping coefficients at yield and ultimate load (β_y and β_u). For these specimens with shear reinforcement the stiffness at ultimate load was about 30 to 40 percent of the stiffness at first yield. Ductility ratios at ultimate varied from 3 up to 6 with values increasing as the reinforcement ratio for the slab decreased. For these specimens with shear reinforcement ultimate ductility ratios were two to three times greater than the ultimate ductility ratios for companion specimens without shear reinforcement. The damping coefficients at yield and ultimate ranged from 8 to 14 percent. The ultimate damping coefficients increase as the reinforcement ratio for the slab decreases.

For specimen SS5 with adequate shear reinforcement the hysteretic loops did not exhibit the pinching effect associated with shear decay of the energy dissipating mechanism. In contrast for SS1 and SS4 the hysteretic loops exhibited marked pinching with cycling.

Also listed in Table 2 are the measured west lateral loads for first yield of the top steel (P_{yt}) and first yield of the bottom steel (P_{yb}) passing through the column, and the ultimate load (P_u). Those values are compared with the capacities, P_{ACI}, predicted by the procedures specified in ACI 318-71[2], with the capacities, P_{BA}, predicted by the beam analogy[4], and the capacities, P_{flex}, for a wide beam flexural mechanism involving yielding of all the bars extending across the width of the slab at the column face.

(2) Proper Detailing of Stirrup Reinforcement

(a) Anchorage of Stirrups - The results of these tests on specimens subject to reversed cyclic loadings and the results for monotonic loading tests on connections with shear reinforcement and identical proportions[5] permit examination of the effects of various stirrup details. Shown in Fig. 6 are three types of closed stirrups employed as shear reinforcement. It was found that for proper anchorage with No. 2 (6.35 mm. diameter) and No. 3 (9.52 mm. diameter) bars, closed stirrups were used, having 135 degree bends around longitudinal corner bars and bar extensions of 2-1/2 in. (6.35 cm.) beyond the bend. In specimen SS1 stirrups with 135 degree bends and a single horizontal leg were used for one half the specimen while similar stirrups with a double horizontal leg were used for the other half. There was no appreciable difference in the performance of the two halves of the slab and therefore the simpler single leg detail was used for specimens SS2 through SS5. As apparent from Fig. 7 that detail was found satisfactory even after reversed cyclic loadings at high deflections had spalled the concrete cover off the stirrups. In contrast even in the monotonic loading tests, use of the lapped splice detail shown in Fig. 6 in the compression region was found to be unsatisfactory. As apparent from Fig. 8 the tendency of the lapped stirrup legs to kick out caused premature spalling of the concrete and permitted an anchorage pull-out failure for the stirrups.

(b) Length of Shear Reinforcement - The closed stirrup reinforcement was extended out from the column to a distance such that the shear stress, v_u, equal to $V_{u/bd}$ and caused by the lateral and gravity loads acting on one half of the specimen was less than $4\sqrt{f'_c}$ ($1.06\sqrt{f'_c}$ if f'_c is expressed in kg/cm^2) for a critical periphery joining the outer legs of the shear reinforcement for that half of the slab. In addition to that criterion it was also found that if the critical shear periphery approached too close to the column perimeter the concentration of shear stress at the column corner could also initiate a shear failure. The behaviour of specimens SS2 and SS5 are compared in Fig. 9. Those specimens had the same flexural reinforcement and the same size and spacing for the stirrup reinforcement. The only difference was a slightly shorter length for the reinforcement extending out from the column for SS2. Specimen SS2 failed by punching at a lateral load considerably less than that for SS5. The failure of SS2 was initiated by a shear crack that developed close to the column corner. The additional reinforced length for SS5 prevented that undesirable failure and permitted development at ultimate of all the flexural reinforcement across the full width of the slab. As a result of the difference in performance for SS2 and SS5 it is recommended that the perimeter joining the outer legs of the shear reinforcement in the slab should not approach closer than 1.5 times the slab thickness, h, to the column perimeter.

(3) Residual Shear Capacity

After the reversed cyclic loading tests were completed residual shear capacity tests were made for several of the specimens by actuating only the gravity load jacking system shown in Fig. 1. Two typical results are shown in Fig. 10. While specimen S4 contained no shear reinforcement it had flexural reinforcement the same as that provided in specimen SS1. Specimen SS3 had shear reinforcement and an average top reinforcement ratio less than that for SS1. However, the amount of reinforcement concentrated within lines 1.5h either side of the column was about double that for S4. Specimen S4 had a residual shear capacity only 30 percent greater than the dead load shear for the prototype structure. In contrast specimen SS3, even after three cycles of loading to a ductility ratio of 4.0 still had a reserve capacity 2.6 times the dead load shear. In the residual shear capacity test specimen SS3 finally collapsed as a result of a diagonal tension crack extending across the width of the slab outside the end of the shear reinforcement.

(4) Concentration of Flexural Reinforcement in Column Region

Specimens SS3 and SS4 had flexural reinforcement concentrated within a 37 in. (94 cm.) wide zone centered on the column. Comparison of Figs. 2 and 4 shows that the concentration of the flexural reinforcement did not markedly improve either the strength or the ductility of SS4 compared to SS1. However, for specimen SS3 subjected to a less severe loading history than SS4 there was some improvement. Comparison of these results with those for SS2 and SS5 with lesser flexural reinforcement shows that there is definite limit to the amount of reinforcement that is effective when concentrated within the column region. In Reference 4, it is concluded that unless the development length of the top reinforcement is less than the column length in the direction of lateral loading the sum of the top and bottom reinforcements with lines $d/2$ either side of the column and considered effective for transfer of moment to the column should not exceed the balanced reinforcement ratio for a section containing tensile reinforcement only. For that concept excess steel was concentrated within the column region for specimens SS3 and SS4.

CONCLUSIONS

(1) A flat plate to column connection will behave in a ductile manner and have adequate residual shear strength if properly

designed and detailed stirrup reinforcement is provided in the slab.

(2) The beneficial effects of providing properly designed and detailed shear reinforcement in the slab are: -

(a) an increase in the ductility of the connection at ultimate load,

(b) an increase in the energy absorption of the connection along with a decrease in stiffness with cycling,

(c) a change in the hysteretic behaviour of connections with low reinforcement ratios from a shear to a moment type of energy dissipation mechanism,

(d) an increase in the strength particularly for low reinforcement ratios,

and (e) an increase in the residual shear capacity.

(3) In order for the stirrups to be fully effective they had to be detailed so that: -

(a) They were closed hoops with a longitudinal reinforcing bar in each corner.

(b) They were anchored by 135 degree standard bends around one or more longitudinal bars.

and (c) They extended far enough out from the column face into each column strip that the wide beam shear force V_u on a perimeter joining the outer legs of the stirrups for one column strip does not result in a shear stress $V_{u/bd}$ exceeding $4\sqrt{f'_c}$ ($1.06\sqrt{f'_c}$ if f'_c is expressed in kg/cm^2) and that perimeter did not approach closer than 1.5h to the column perimeter.

(4) The behaviour of the connection, especially for low reinforcement ratios, is likely to be improved if the flexural reinforcement is concentrated around the immediate column region.

(5) The strength of connections containing adequate shear reinforcement can be evaluated by either the ACI 318-71 procedure[2] or the beam analogy[4]. Both give approximately the same predictions, are conservative for the lower reinforcement ratios and unconservative for the higher reinforcement ratios.

REFERENCES

1. ACI-ASCE Committee 426 "The Shear Strength of Reinforced Concrete Members-Slabs", Journal of the Structural Division, ASCE, Vol. 100, No. ST8, Aug. 1974.

2. ACI Committee 318, "Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71)", Detroit, Michigan, 1971.

3. Hawkins, N. M., Mitchell, D. and Sheu, M. S., "Reversed Cyclic Loading Behavior of Reinforced Concrete Slab-Column Connections", to be published in Proceedings of U.S. National Conference on Earthquake Engineering, Ann Arbor, Michigan, June 1975.

4. Hawkins, N. M., "Shear Problems in Slabs", Preprint, ASCE National Structural Convention, New Orleans, April 1975.

5. Yamazaki, J., "Shear and Moment Transfer Between Reinforced Concrete Flat Plates and Column", Ph. D Thesis, University of Washington, March 1975.

TABLE 1 PROPERTIES OF TEST SPECIMENS

Specimen	Concrete Strength Slab (top column) p.s.i.	Top Bars Number; Size; Spacing Percent; Yield Strength	Bottom Bars Number; Size; Spacing Percent; Yield Strength	Cyclic History No. of Cycles at Given Load or Ductility Level	Constant Gravity Load, P_D kips	Shear Reinforcement
SS1 *(S1 & S4)	4000 (3280)	12; No. 6; 7-1/2 in., 1.29%; 66.6 ksi	12, No. 4, 7-1/2 in., 0.59%, 66.0 ksi	1c@±3.2k; 1c@±4.5k; 1c@±5.3k; 5c@±μ=1.0; 5c@±μ=1.3; 5c@±μ=1.7; 5c@±μ=2.0; 1c@±μ=2.5; 4c@±μ=2.9; 9c@±μ=3.0; 1c@±μ=3.3	29.9	No. 3 stirrups with 68.0 ksi yield stress at 1-1/2 in. spacing to 15.75 in. from each column face. Loading history as for S4.
SS2 *(S2)	3730 (3700)	12; No. 5; 7-1/2 in.; 0.90%; 67.1 ksi	10; No. 4; 9 in.; 0.49%; 66.0 ksi	5c@±4.8k; 1c@±5.5k; 6c@±μ=3.0	28.4	No. 2 stirrups with 65.8 ksi yield stress at 1-1/2 in. spacing to 11.25 in. from each column face.
SS3 *(S5)	3750 (3850)	8 No. 6 at 5 in. for central 36 in; 6 No. 4 at 8	8 No. 4 at 5 in. for central 36 in; 6 No. 3 at 8	3 cycles each @±μ =0.62;1.0;2.0;3.0;4.0	28.5	No. 3 stirrups with 68.0 ksi yield stress

		in. for outside region; 1.1%; No. 6 bars 66ksi No. 4 bars-66.0 ksi	in. for outside region; 0.56%; No. 4 bars-66.0 ksi No. 3 bars-68.0 ksi			at 1-1/2 in. spacing to 14.25 in. from each column face.
SS4 *(S5)	4000 (4530)	as for SS3	as for SS3	3c@±μ=0.62;11c@±μ=2.1; 2c@±μ=1.5; 2c@±μ=3.0	28.7	Stirrups as for SS3 loading history with large no. of cycles at ±μ= 2.1 .
SS5 *(S2)	4670 (2630)	as for SS2	as for SS2	3c@±μ=.81; 3c@±μ=1.6; 3c@±μ=2.5; 5c@±μ=4.1	28.3	No. 2 stirrups with 65.8 ksi yield stress at 1-1/2 in. spacing to 12.75 in. from each column face.

* Indicates companion specimens without shear reinforcement having the same amount and distribution of flexural reinforcement.

Metric conversion factors: - 1 p.s.i. = .07031 kgf/cm^2, 1 in. = 2.54 cm., 1 kip = 453.6 kgf

TABLE 2 - TEST RESULTS

Specimen	Measured Lateral Loads (1 kip=453.6 kgf.) P_{yt} kips	P_{yb} kips	P_u kips	Predicted Lateral Loads (1 kip=453.6 kgf.) P^1_{ACI} kips	P^2_{flex} kips	P_{BA} kips	Response Charactristics λ_y p.s.i. x10^{-6} (1p.s.i.=.07031 kgf/cm^2)	$\frac{\lambda_u}{\lambda_y}$	μ_u	β_y %	β_u %
SS1	7.0	-5.3	9.8	9.2	16.8	10.0	536	0.41	3.3	9.7	8.0
SS2	4.8	-4.0	6.9[3]	7.6	11.2	7.5	465	0.27	4.0	9.9	14.0
SS3[5]	5.9	-8.8	11.3	12.4	14.5	13.1	-	-	-	-	-
SS4	6.5	-6.4	9.3	12.7	14.7	13.1	550	.35	4.8	9.5	11.9
SS5	4.5	-7.9	9.3[4]	7.7	11.5	7.5	471	.27	5.9	9.5	14.2

[1] ACI "Moment Cut-off" predictions. Shear reinforcement was provided to develop flexural reinforcement.

[2] West lateral load to cause flexural yielding across full specimen width.

[3] Specimen failed prematurely due to insufficient length of stirrup reinforcement.

[4] Flexural reinforcement developed across full width of specimen.

[5] Some values are not shown because results are not completely reduced.

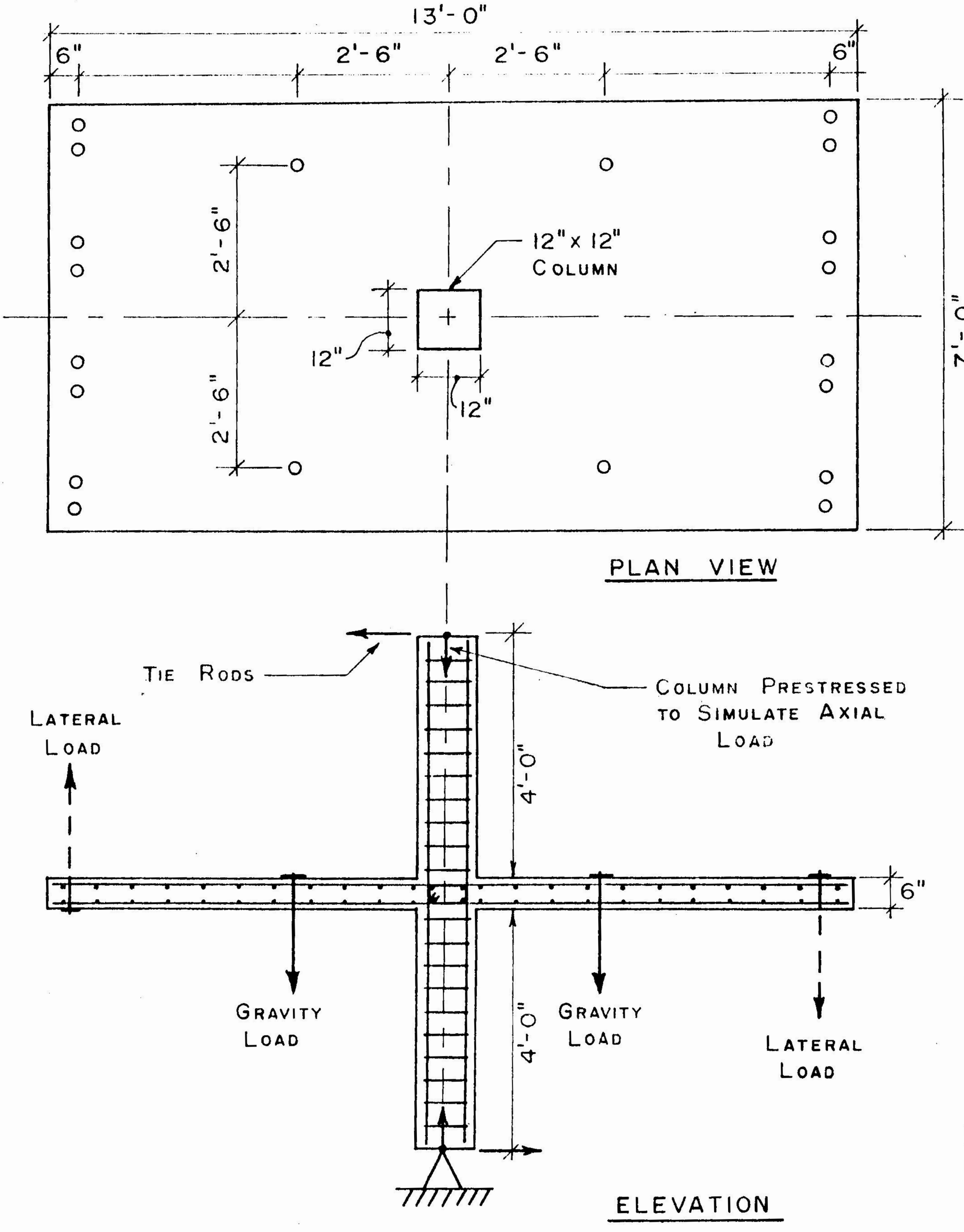


FIG. 1 TEST SPECIMEN AND LOADING ARRANGEMENT
(1 IN. = 2.54 CM, 1 FT. = .3048 M.)

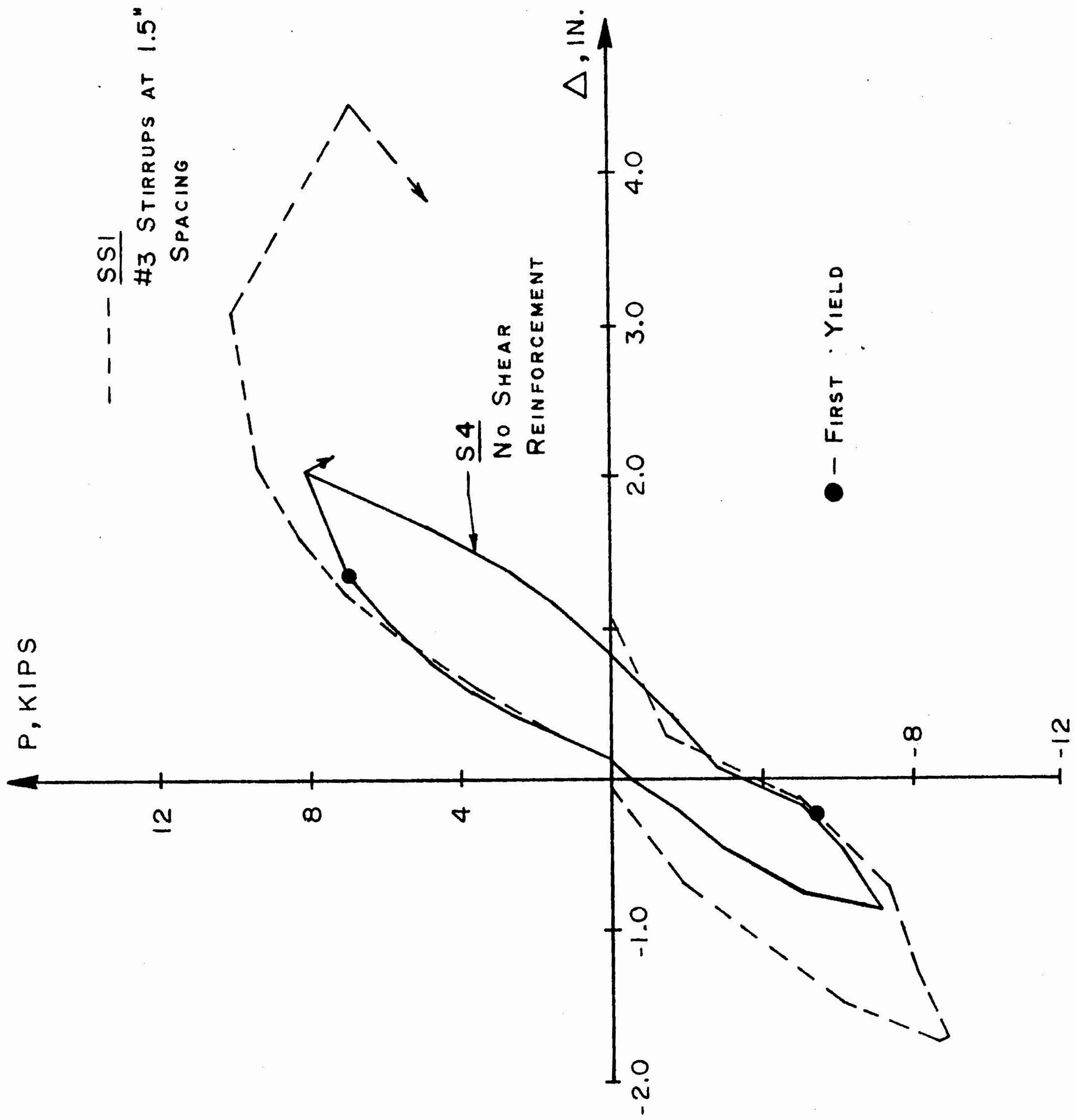


FIG. 2 LATERAL LOAD VS. SLAB END DEFLECTION (P-Δ) ENVELOPES FOR SPECIMEN S4 (WITHOUT SHEAR REINFORCEMENT) AND SS1 (WITH SHEAR REINFORCEMENT) BOTH SPECIMENS HAVING THE SAME FLEXURAL REINFORCEMENT, ρ = 1.29%
(1 KIP = 453.6 KGF, 1 IN. = 2.54 CM.)

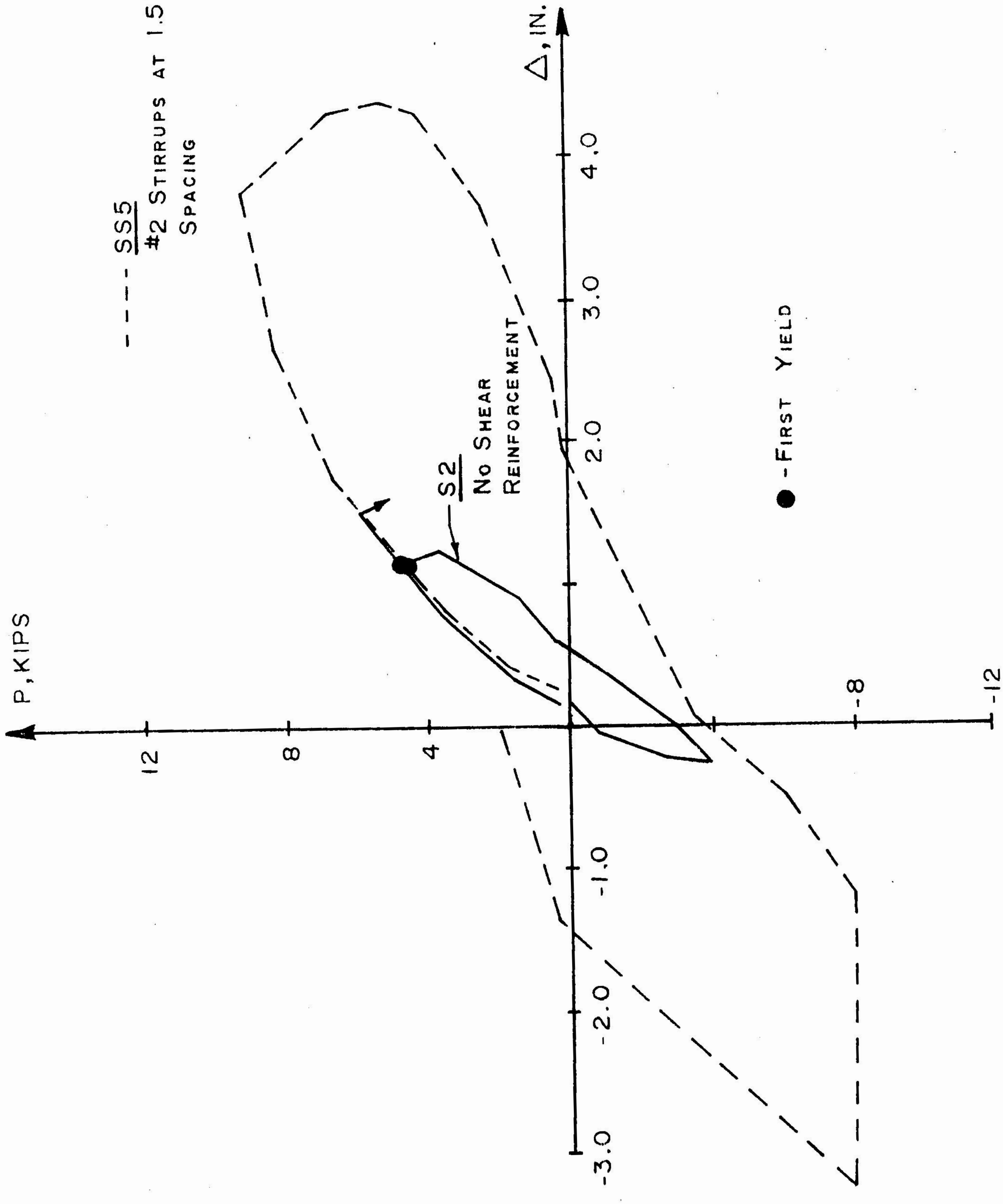


FIG. 3 LATERAL LOAD VS. SLAB END DEFLECTION (P-Δ) ENVELOPES FOR SPECIMEN S2 (WITHOUT SHEAR REINFORCEMENT) AND SS5 (WITH SHEAR REINFORCEMENT) BOTH SPECIMENS HAVING THE SAME FLEXURAL REINFORCEMENT, ρ = 0.90%
(1 KIP = 453.6 KGF, 1 IN. = 2.54 CM.)

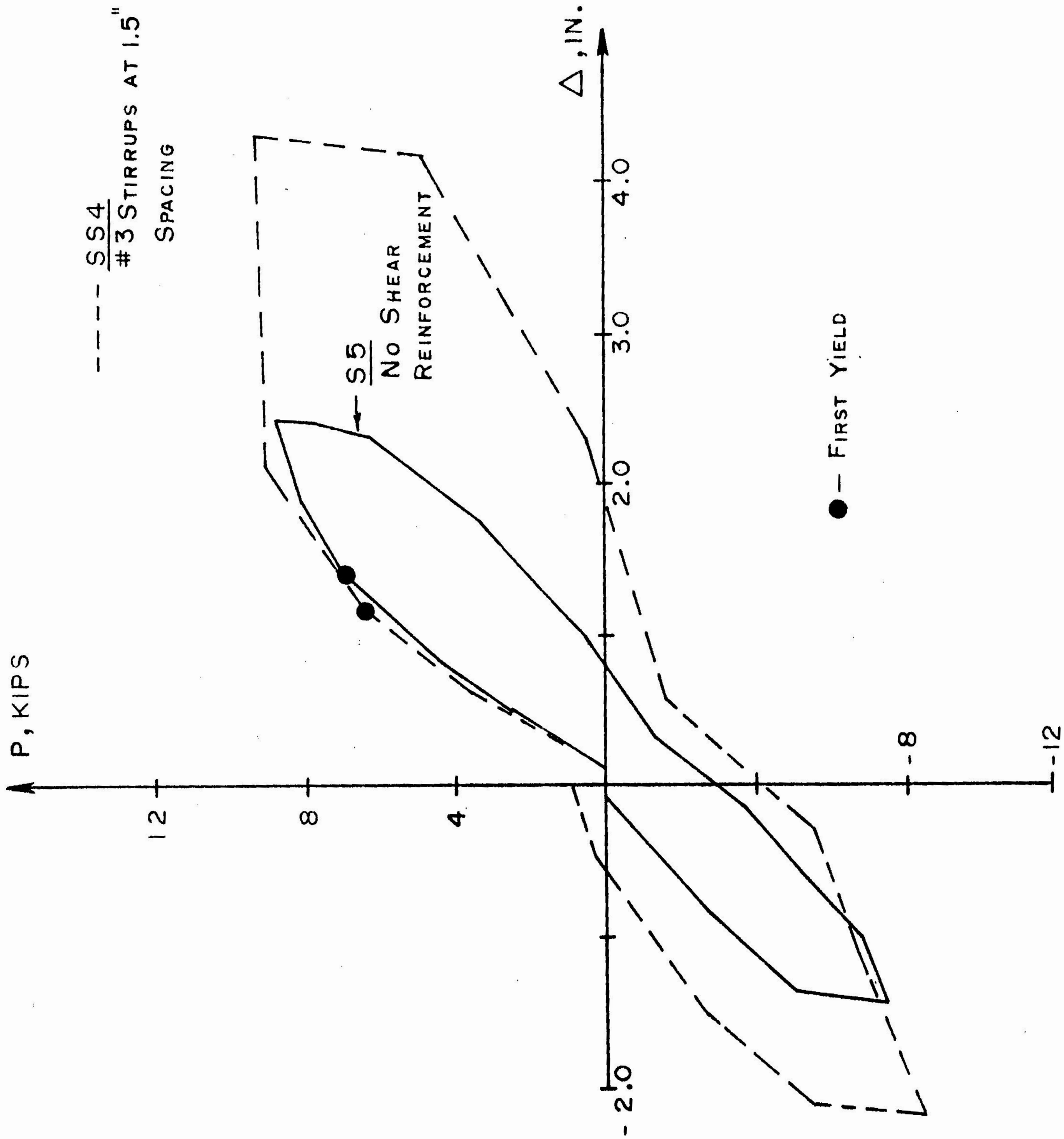


FIG. 4 LATERAL LOAD VS. SLAB END DEFLECTION (P-Δ) ENVELOPES FOR SPECIMEN S5 (WITHOUT SHEAR REINFORCEMENT) AND SS4 (WITH SHEAR REINFORCEMENT) BOTH SPECIMENS HAVING THE SAME CONCENTRATED FLEXURAL REINFORCEMENT (1 KIP = 453.6 KGF, 1 IN. = 2.54 CM.)

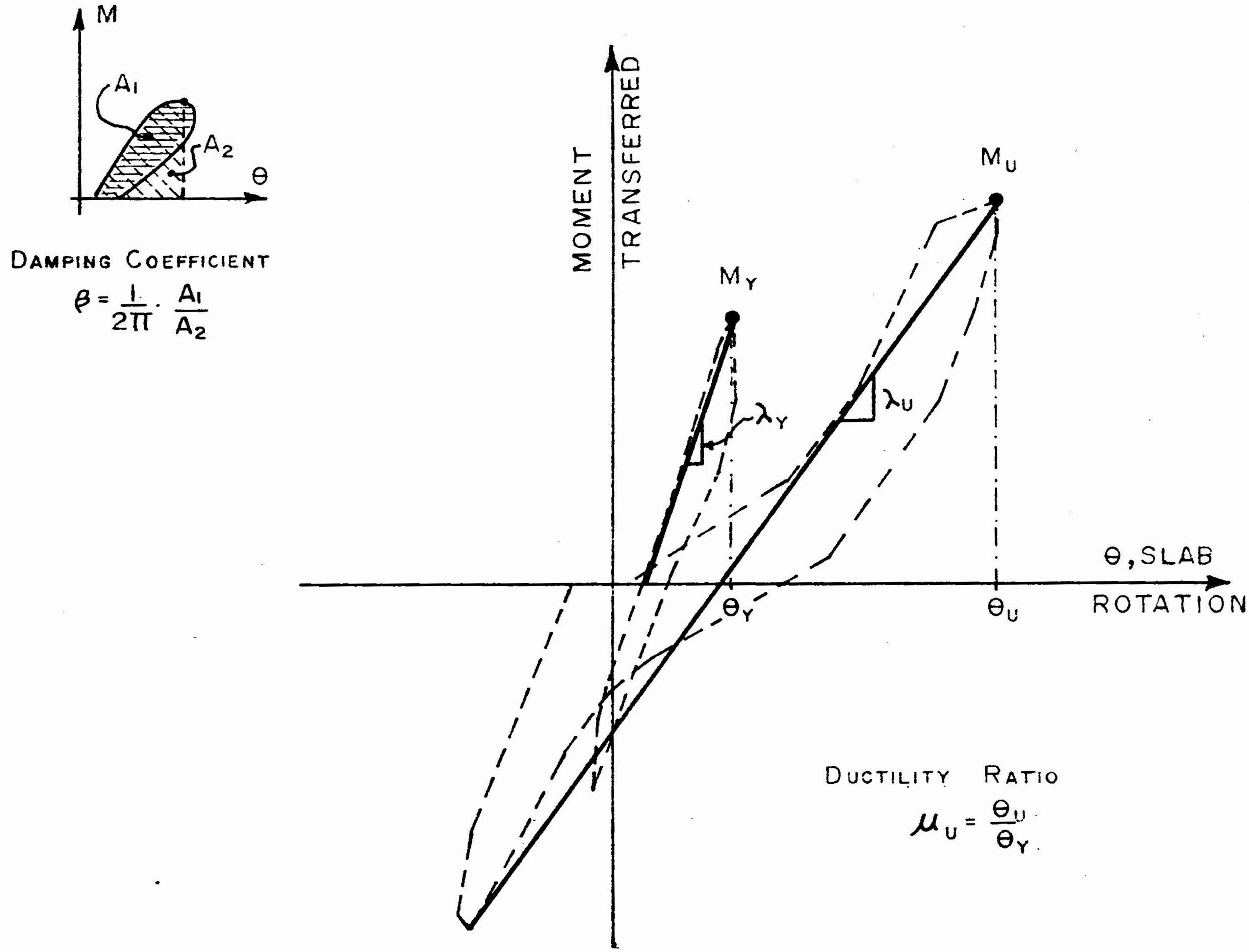
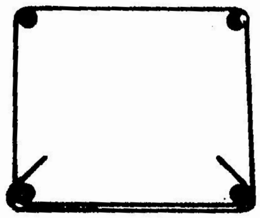
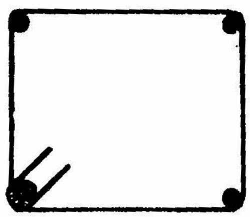


FIG. 5 SEISMIC RESPONSE PARAMETERS FROM MOMENT-SLAB ROTATION (M-θ) CYCLES

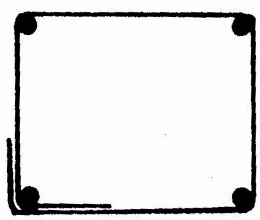


-135° BENDS WITH DOUBLE HORIZONTAL LEG



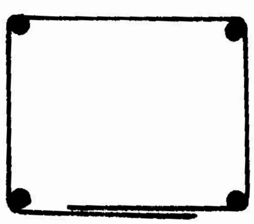
-135° BENDS

PROPER STIRRUP DETAILS

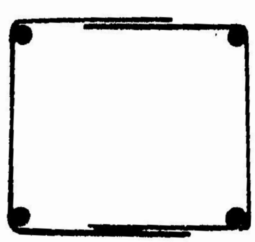


-90° BENDS

FIG. 6 PROPER AND IMPROPER STIRRUP DETAILS



-LAP SPLICE



-DOUBLE LAP SPLICE

IMPROPER STIRRUP DETAILS

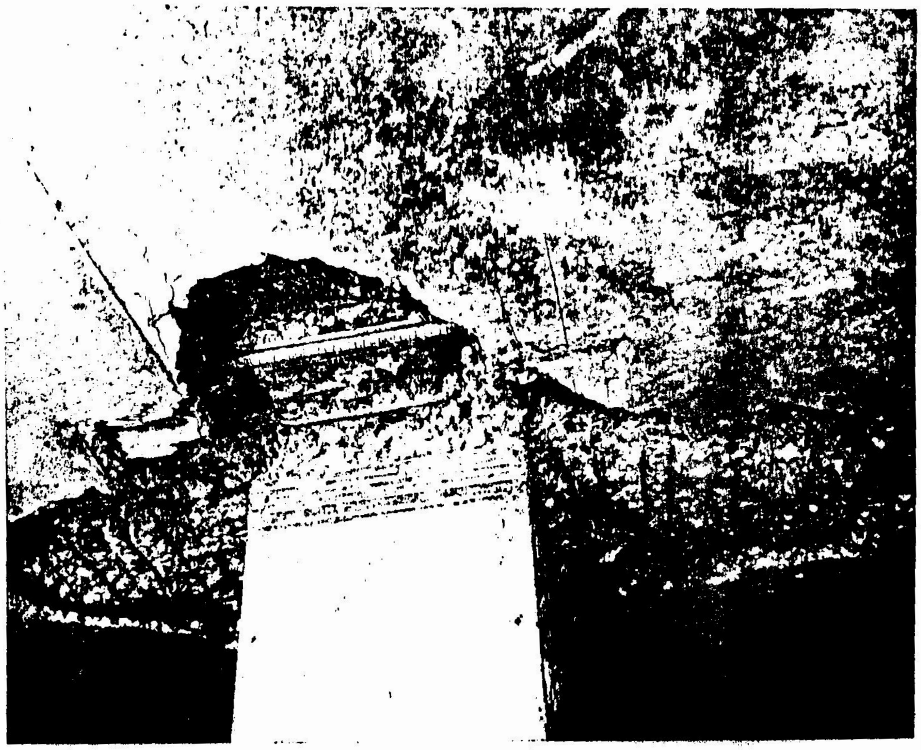


FIG. 7 SPALLING OF CONCRETE COVER ON BOTTOM OF SLAB FOR SPECIMEN SS5 HAVING STIRRUPS ANCHORED WITH 135 DEGREE BENDS

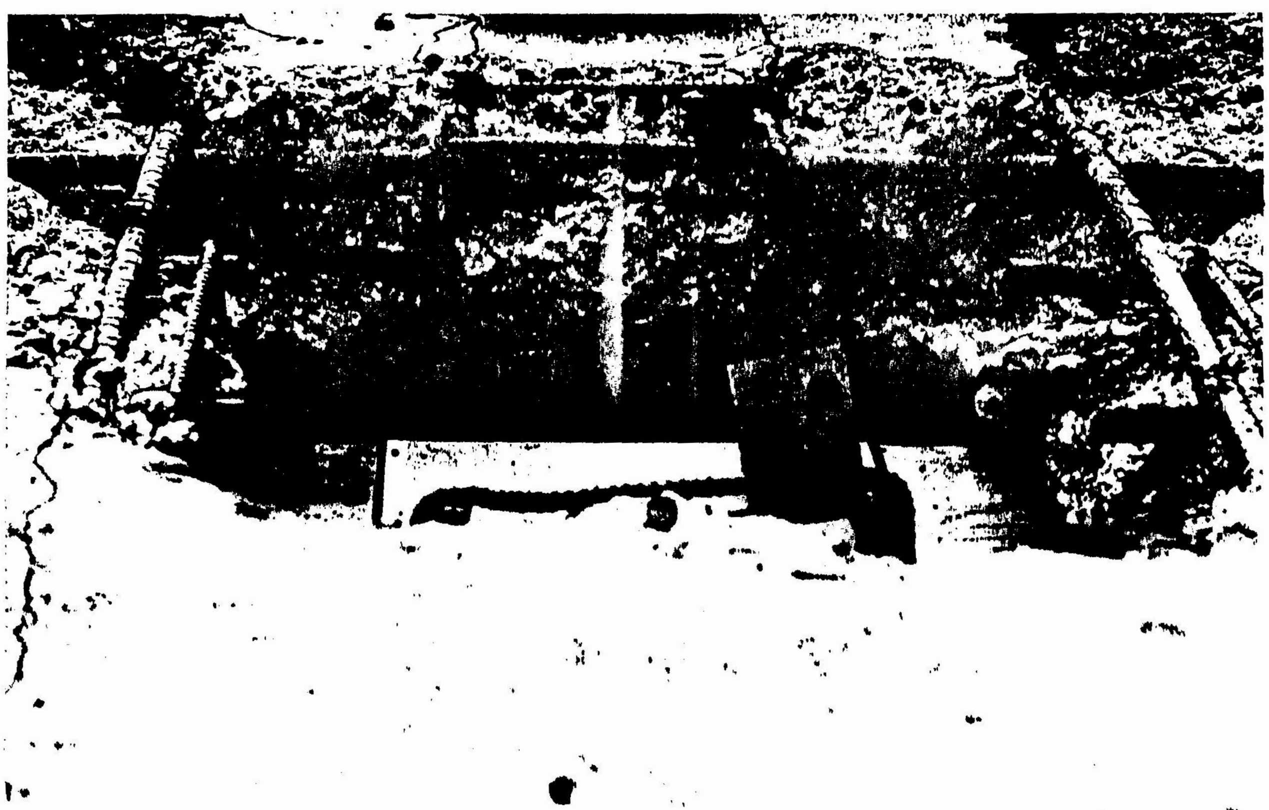


FIG. 8 LAP SPLICE FAILURE IN STIRRUP CLOSE TO COLUMN

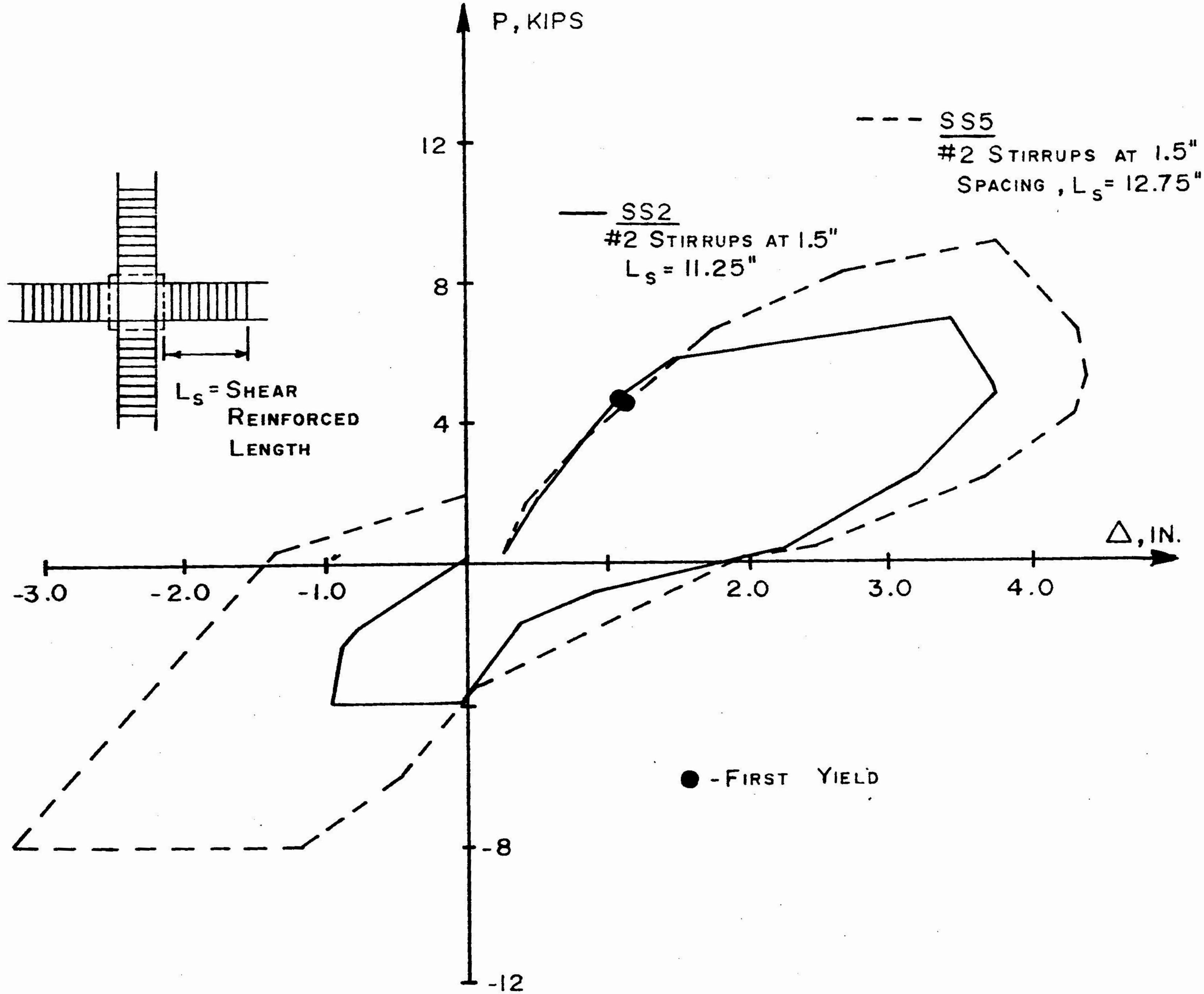


FIG. 9 LATERAL LOAD VS. DEFLECTION ENVELOPES FOR SPECIMENS SS2 AND SS5 (1 KIP = 453.6 KGF, 1 IN. = 2.54 CM.)

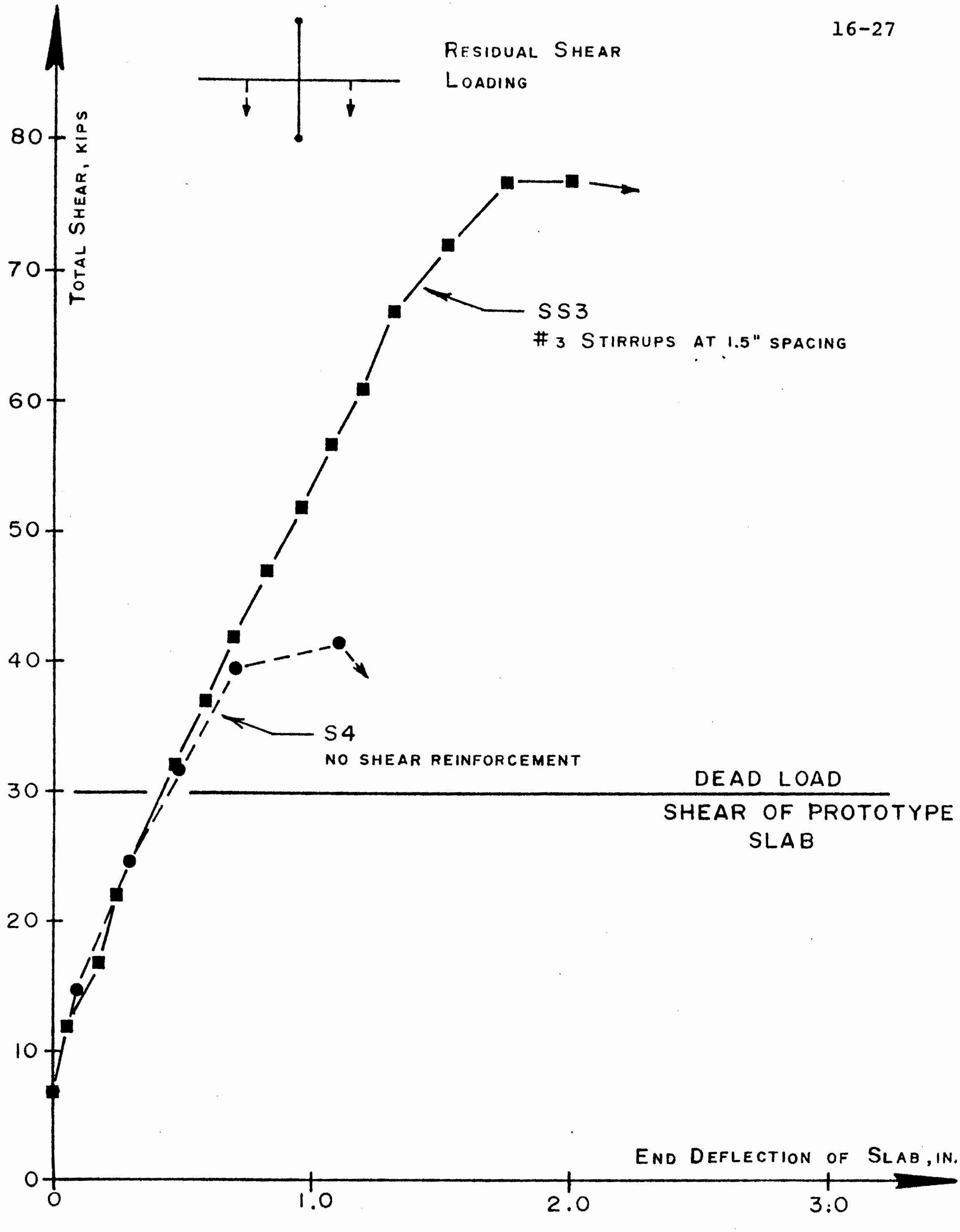


FIG. 10 RESIDUAL SHEAR VS. DEFLECTION BEHAVIOUR FOR SPECIMENS S4 AND SS3 (1 KIP = 453.6 KGF, 1 IN. = 2.54 CM.)