The Effects of Shear Reinforcement on the Reversed Cyclic Loading Behavior of Flat Plate Structures[1]

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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 behavior of connections containing integral beam stirrup reinforcement in the slabs is discussed. The paper compares different approaches for designing that stirrup reinforcement, outlines the benefits of placing stirrup reinforcement in the slab, and gives guidelines as to the design and proper detailing necessary for that stirrup 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.

Cet article décrit une phase d'un programme de recherches menées à l'université de Washington sur la réponse en régime sismique d'une dalle sous champignons en fonction du type de liaisons qui la relient aux poteaux. On expose en particulier le comportement des liaisons contenant des armatures en étriers dans les dalles. Les auteurs comparent différentes méthodes de calcul de ces étriers, indiquent les avantages qu'il y a à pourvoir les dalles de tels étriers pour lesquels par ailleurs ils fournissent des règles générales de calcul et proposent une méthode de description détaillée (répartition et dessin). Ils examinent en outre les effets qu'entraîne la concentration des armatures de flexion dans la zone à proximité d'un poteau. L'article contient des éclaircissements sur les rapports de ductilité, l'absorption d'énergie, la dissipation d'énergie, la dégradation progressive de la rigidité, la grandeur des charges latérales amorçant la plastification et la capacité maximale des spécimens étudiés.

[Traduit par la Revue]

Introduction

Primarily because of recent collapses during earthquakes and the known lack of ductility and energy absorption of slab–column connections (American Concrete Institute – American Society of Civil Engineers 1974), any flat plate framing is generally neglected for evaluations of the seismic resistance of concrete structures (American Concrete Institute 1971). 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 or shear walls. During an earthquake, there is a possibility of the slab–column connections failing and contributing significantly to the damage of the structure. A test program in progress at the University of Washington indicates that major damage can readily be avoided by the provision of carefully detailed stirrup reinforcement in the slabs. This work confirms findings in other tests conducted at the Portland Cement Association (Hanson and Hanson 1968) and the University of Canterbury, New Zealand (Islam 1973). In these previous tests the effects of shearhead and integral beam reinforcement have been compared and it was concluded that only integral beam stirrup reinforcement ensured satisfactory performance for reversed cyclic loading. In this paper results are reported of tests on five slab–column specimens containing integral beam stirrup reinforcement. The results of those tests are compared with the results of tests on similar specimens without shear reinforcement (Hawkins *et al.* 1975).

Simulation of Slab–Column Connection

Specimen dimensions were chosen so as to permit a realistic examination of the behavior of slab–column connections under constant dead load coupled with reversed cyclic lateral

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loads. 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. (15.24 cm) 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 and 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 setup is shown in Fig. 1 and idealization of the specimen from the prototype structure is discussed in greater detail in the references (Hawkins *et al.* 1975).

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 (the west and the east end) and were connected to a second common pumping source.

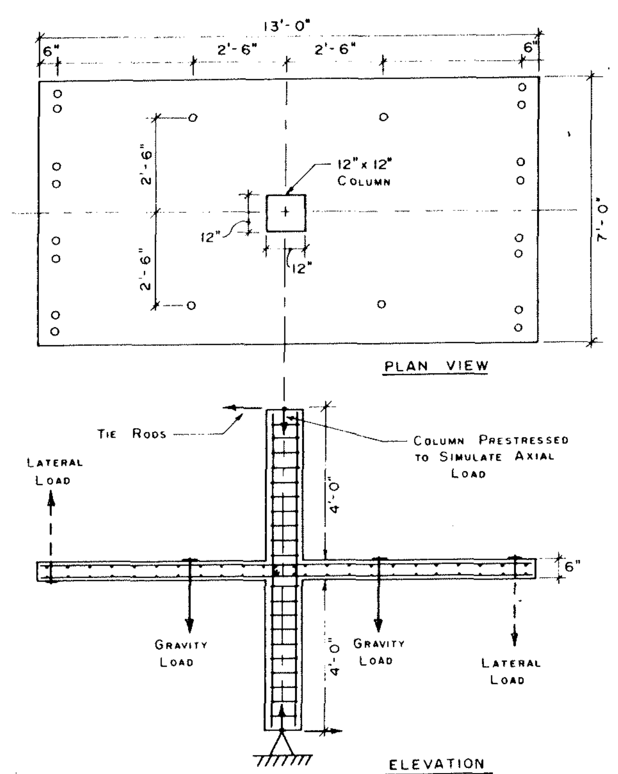

FIG. 1. Test specimen and loading arrangement (1 in. = 2.54 cm, 1 ft = 0.3048 m).

Experimental Program

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

(1) The size, spacing, and length of integral beam stirrup reinforcement necessary in the slab to provide a ductile behavior 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 gravity load capacity of connections that have undergone reversed cyclic loadings comparable to those likely in a major earthquake.

(4) The effects of concentrating the flexural reinforcement around the column, within a distance of 1.5 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 Hawkins *et al.* (1975). 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 American Concrete Institute (ACI) Code provisions (1971) 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 behavior throughout its entire loading history. The applied gravity loads and the lateral loads were monitored by means of load cells positioned

TABLE 1. Properties of test specimens

Specimen	Concrete strength slab (top column) (p.s.i.)	Top bars (No.; size; spacing %; yield strength)	Bottom bars (No.; size; spacing %; yield strength)	Cyclic history (No. of cycles at given load or ductility level)	Constant gravity load, P_D (kips)	Shear reinforcement
SS1 (S1 and S4)[a]	4000 (3280)	12; No. 6; 7.5 in.; 1.29%; 66.6 k.s.i.	12, No. 4, 7.5 in.; 0.59%, 66.0 k.s.i.	1c at ± 3.2k; 1c at ± 4.5k; 1c at ± 5.3k; 5c at $\pm \mu = 1.0$; 5c at $\pm \mu = 1.3$; 5c at $\pm \mu = 1.7$; 5c at $\pm \mu = 2.0$; 1c at $\pm \mu = 2.5$; 4c at $\pm \mu = 2.9$; 9c at $\pm \mu = 3.0$; 1c at $\pm \mu = 3.3$	29.9	No. 3 stirrups with 68.0 k.s.i. yield stress at 1.5 in. spacing to 15.75 in. from each column face. Loading history as for S4
SS2 (S2)[a]	3730 (3700)	12; No. 5; 7.5 in.; 0.90%; 67.1 k.s.i.	10; No. 4; 9 in.; 0.49%; 66.0 k.s.i.	5c at ± 4.8k; 1c at ± 5.5k; 6c at $\pm \mu = 3.0$	28.4	No. 2 stirrups with 65.8 k.s.i. yield stress at 1.5 in. spacing to 11.25 in. from each column face
SS3 (S5)[a]	3750 (3850)	8 No. 6 at 5 in. for central 36 in.; 6 No. 4 at 8 in. for outside region; 1.1%; No. 6 bars 66 k.s.i., No. 4 bars 66.0 k.s.i.	8 No. 4 at 5 in. for central 36 in.; 6 No. 3 at 8 in. for outside region; 0.56%; No. 4 bars 66.0 k.s.i., No. 3 bars 68.0 k.s.i.	3c at $\pm \mu = 0.62$; 1.0; 2.0; 3.0; 4.0	28.5	No. 3 stirrups with 68.0 k.s.i. yield stress at 1.5 in. spacing to 14.25 in. from each column face
SS4 (S5)[a]	4000 (4530)	As for SS3	As for SS3	3c at $\pm \mu = 0.62$; 11c at $\pm \mu = 2.1$; 2c at $\pm \mu = 1.5$; 2c at $\pm \mu = 3.0$	28.7	Stirrups as for SS3 loading history with large no. of cycles at $\pm \mu = 2.1$
SS5 (S2)[a]	4670 (2630)	As for SS2	As for SS2	3c at $\pm \mu = 0.81$; 3c at $\pm \mu = 1.6$; 3c at $\pm \mu = 2.5$; 5c at $\pm \mu = 4.1$	28.3	No. 2 stirrups with 65.8 k.s.i. yield stress at 1.5 in. spacing to 12.75 in. from each column face

[a]Indicates companion specimens without shear reinforcement having the same amount and distribution of flexural reinforcement.
NOTE: Metric conversion factors: 1 p.s.i. = 0.07031 kgf/cm^2, 1 in. = 2.54 cm, 1 kip = 453.6 kgf.

at the loading points indicated on Fig. 1. A combination of linear potentiometers, dial gauges, and deflection scales was 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

The complete lateral load–slab deflection relationships for the five specimens tested are shown in Fig. 2. Indicated on the plots are the first yield of the top reinforcement and the first yield of the bottom reinforcement. It can be seen from Fig. 2 that in general at least three cycles were applied after each increment of load and before the deflection was again incremented.

(1) The Benefits of Shear Reinforcement

In Figs. 3, 4, and 5 west lateral load *versus* specimen edge deflection envelopes are compared for specimens with and without 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 as shown in Fig. 1.

The benefits of shear reinforcement are readily apparent from Figs. 3, 4, and 5. The ductility, energy absorption, and strength 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. 6, from the moment transferred to the column *versus* 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 (λ_u/λ_y), the ductility ratio at ultimate load (μ_u), and 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% 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%. 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 as can be seen in Fig. 2.

Also listed in Table 2 are the measured west lateral loads for first yield of the top steel (P_{yt}) the 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_{BA}, predicted by the beam analogy (Hawkins 1975), 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 (Yamazaki 1975) permit examination of the effects of various stirrup details. Shown in Fig. 7 are several types of closed stirrups

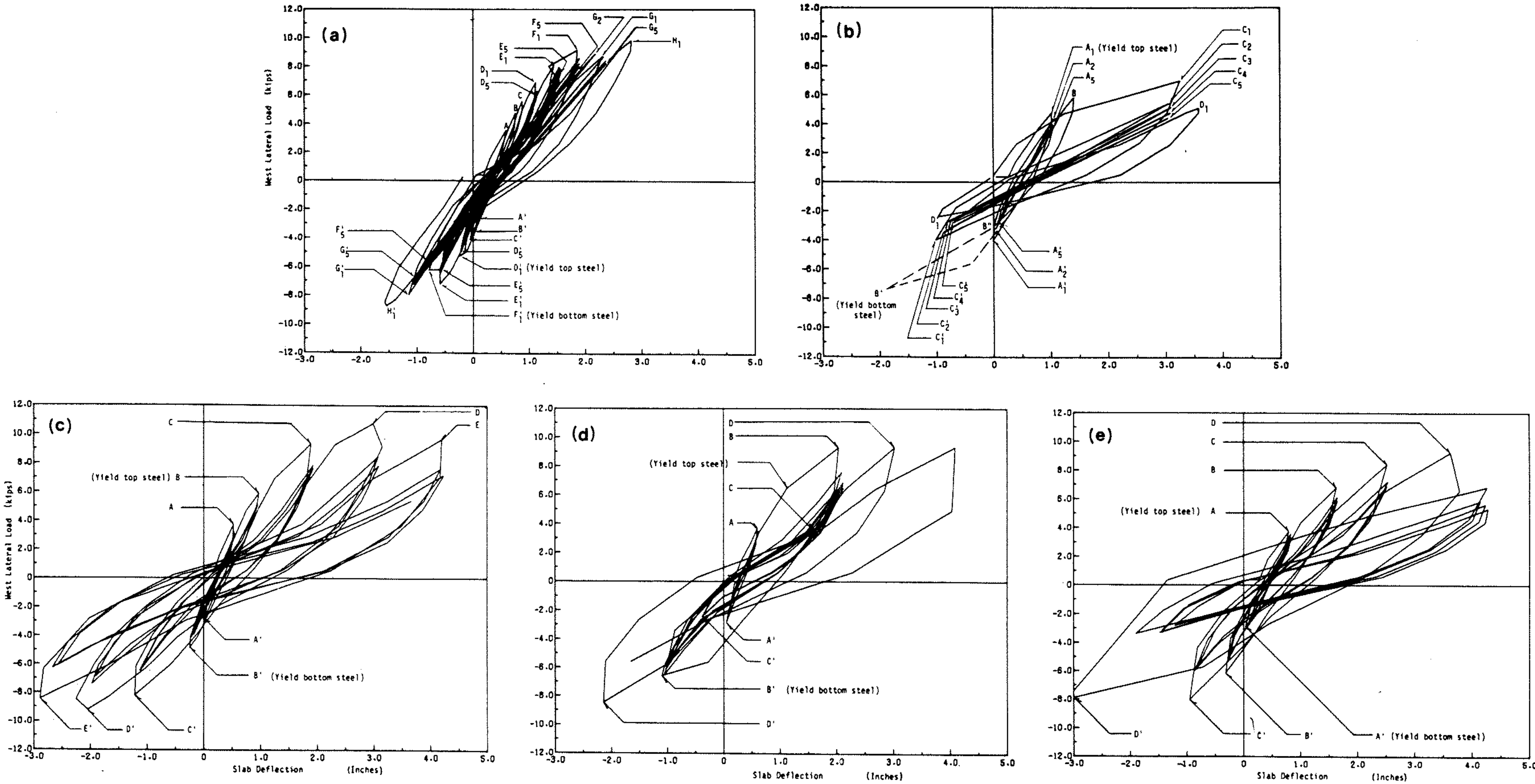

FIG. 2. Complete lateral load *versus* slab end deflection relationships (1 kip = 453.6 kgf, 1 in. = 2.54 cm). (a) West side SS1, (b) west side SS2, (c) west side SS3, (d) west side SS4, (e) west side SS5.

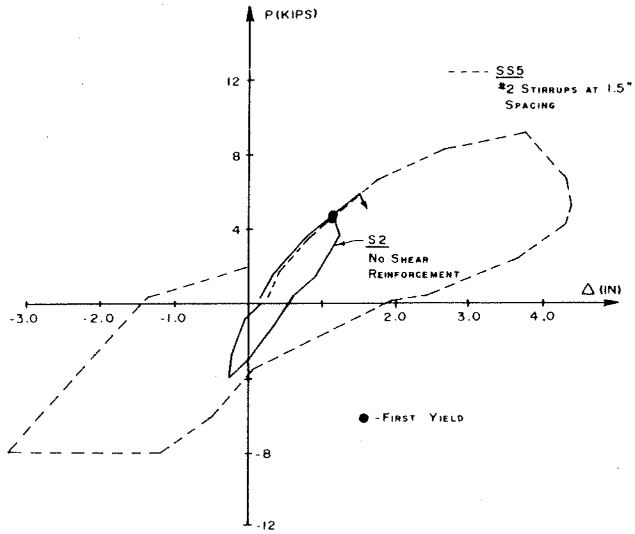

FIG. 3. Lateral load *versus* slab end deflection (P–Δ) envelopes for specimen S4 (without shear reinforcement) and SS1 (with shear reinforcement) both specimens having the same flexural reinforcement, ρ = 1.29% (1 kip = 453.6 kgf, 1 in. = 2.54 cm).

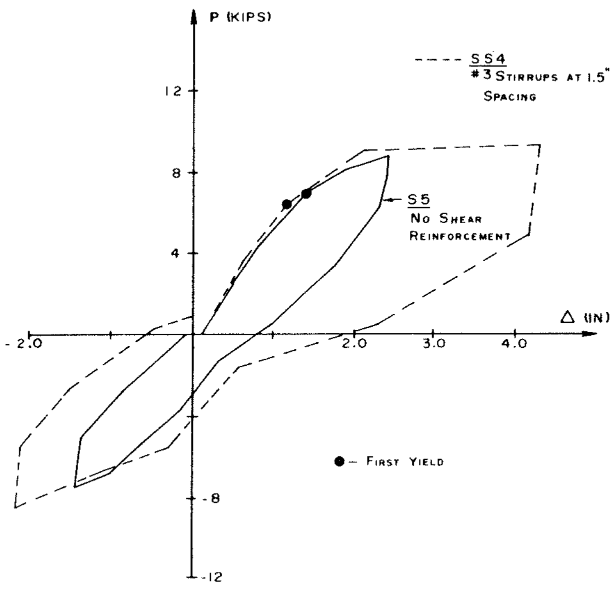

FIG. 4. Lateral load *versus* slab end deflection (P–Δ) envelopes for specimen S2 (without shear reinforcement) and SS5 (with shear reinforcement) both specimens having the same flexural reinforcement, ρ = 0.90% (1 kip = 543.6 kgf, 1 in. = 2.54 cm).

that can be employed as shear reinforcement according to the ACI Code (1971). 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 required, having 135° bends around longitudinal cor-

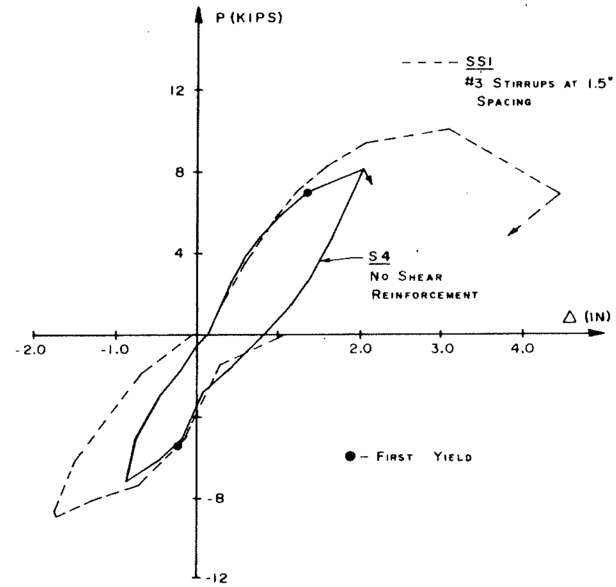

FIG. 5. Lateral load *versus* slab end deflection (P–Δ) envelopes for specimen S5 (without shear reinforcement) and SS4 (with shear reinforcement) both specimens having the same concentrated flexural reinforcement (1 kip = 543.6 kgf, 1 in. = 2.54 cm).

ner bars and bar extensions of 2.5 in. (6.35 cm) beyond the bend. In specimens SS1 and SS2 stirrups with 135° 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 SS3 through SS5. As apparent from Fig. 8, 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. 7 in the compression region was found to be unsatisfactory. As apparent from Fig. 9, 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) Extent 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)

TABLE 2. Test results

	Measured lateral loads (1 kip = 453.6 kgf)			Predicted lateral loads (1 kip = 453.6 kgf)			Response characteristics				
Specimen	P_{yt} (kips)	P_{yb} (kips)	P_u (kips)	P_{ACI}[a] (kips)	P_{flex}[b] (kips)	P_{BA} (kips)	λ_y (p.s.i. $\times$ 10^{-6}) (1p.s.i. = 0.07031 kgf/cm^2)	λ_u/λ_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[c]	7.6	11.2	7.5	465	0.27	4.0	9.9	14.0
SS3	5.9	−8.8	11.3	12.4	14.5	13.1	660	0.40	3.7	9.8	12.7
SS4	6.5	−6.4	9.3	12.7	14.7	13.1	550	0.35	4.8	9.5	11.9
SS5	4.5	−7.9	9.3[d]	7.7	11.5	7.5	471	0.27	5.9	9.5	14.2

[a]ACI 'moment cut-off' predictions. Shear reinforcement was provided to develop flexural reinforcement.
[b]West lateral load to cause flexural yielding across full specimen width.
[c]Specimen failed prematurely due to insufficient length of stirrup reinforcement.
[d]Flexural reinforcement developed across full width of specimen.

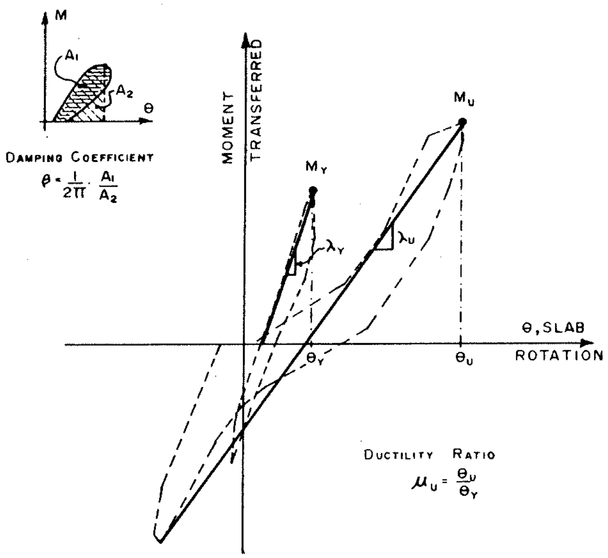

FIG. 6. Seismic response parameters from moment–slab rotation (M–θ) cycles.

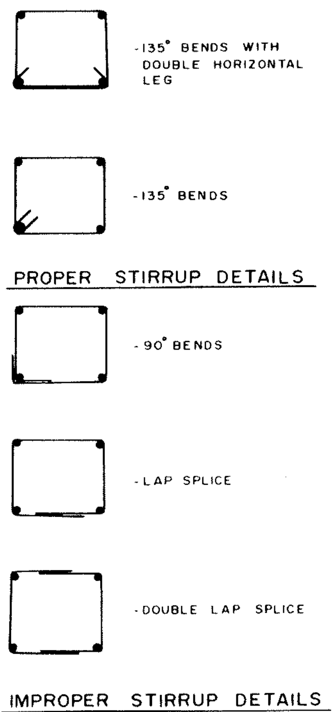

FIG. 7. Proper and improper stirrup details.

for the critical periphery located outside the stirrups as shown in Fig. 10.

It was found in subsequent testing of the specimens that in order to prevent a punching failure around the ends of the stirrups those stirrups had to extend far enough out from the column face that the shear stress caused by an ultimate shear, V_u, equal to $V_D/4 + V_L$ where V_D was the gravity load shear at failure and V_L the lateral load shear at failure on one side of the specimen, was less than $2\sqrt{f_c'}$ ($0.53\sqrt{f_c'}$ if f_c' is expressed in kg/cm^2) for the periphery shown in Fig. 10. Code concepts (American Concrete Institute 1971) would imply that the limiting shear stress should be $4\sqrt{f_c'}$ and the reduction to $2\sqrt{f_c'}$ is probably due to the cyclic nature of the loading. That loading caused flexural cracking through the depth of the slab to a position about midway between the column and the lateral load lines. In addition to the foregoing 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 behavior of specimens SS2 and SS5 is compared in Fig. 11. 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. 12. While specimen S4 contained no shear reinforcement it had flexural reinforcement the same as that provided in specimen SS1. Speciment 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

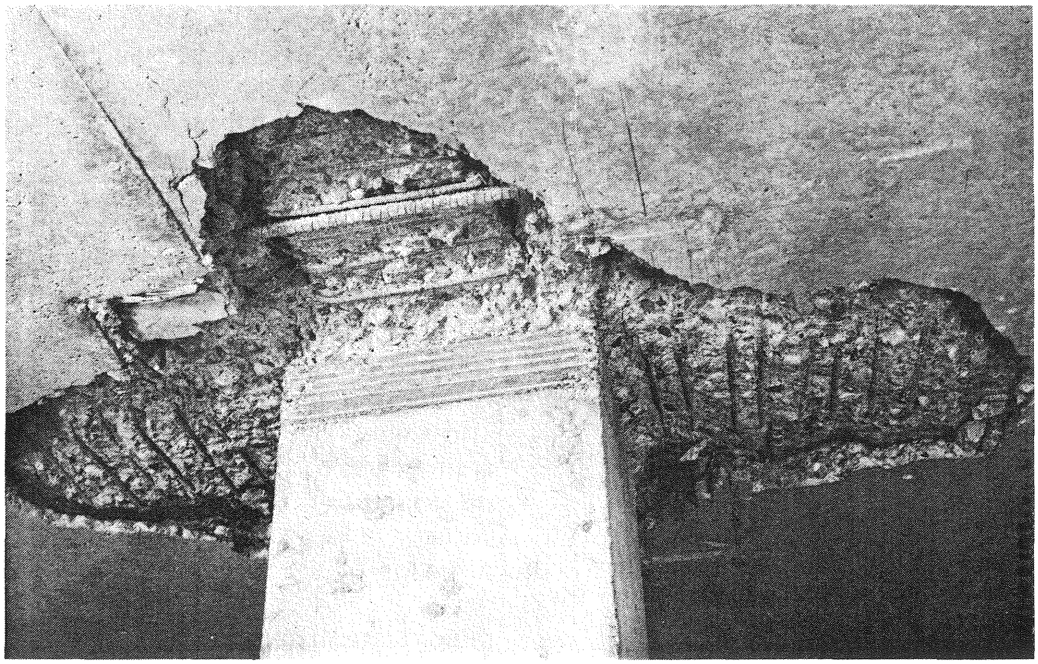

FIG. 8. Spalling of concrete cover on bottom of slab for specimen SS5 having stirrups anchored with 135° bends.

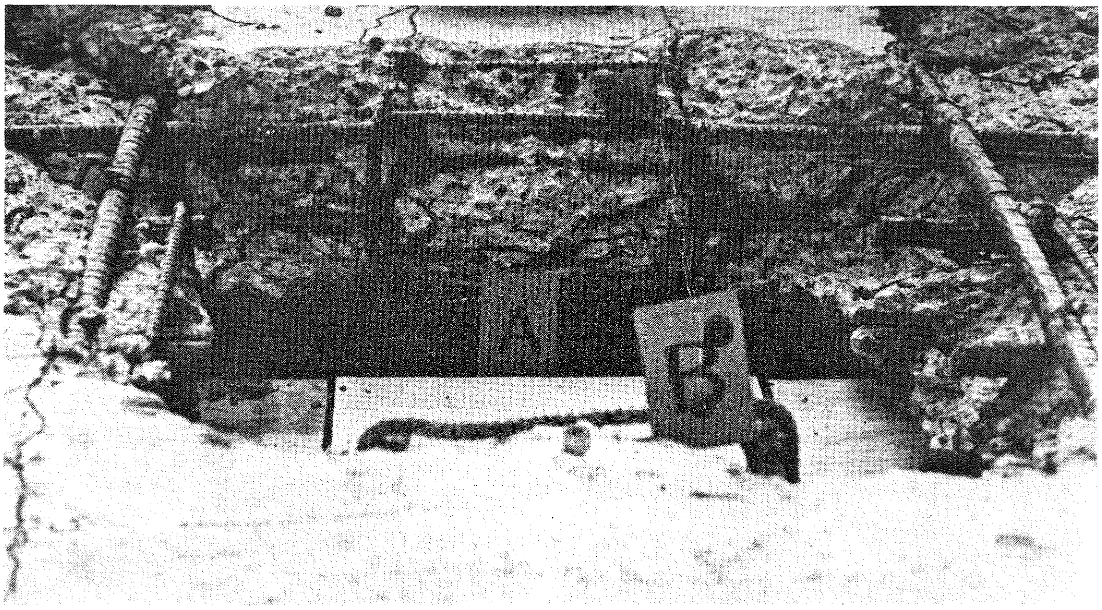

FIG. 9. Lap splice failure in stirrup close to column.

only 30% 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 reverse 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. 3 and 5 shows that the concentration of the flexural reinforcement did not markedly improve either the strength or the ductility of SS4 compared to SS1. However,

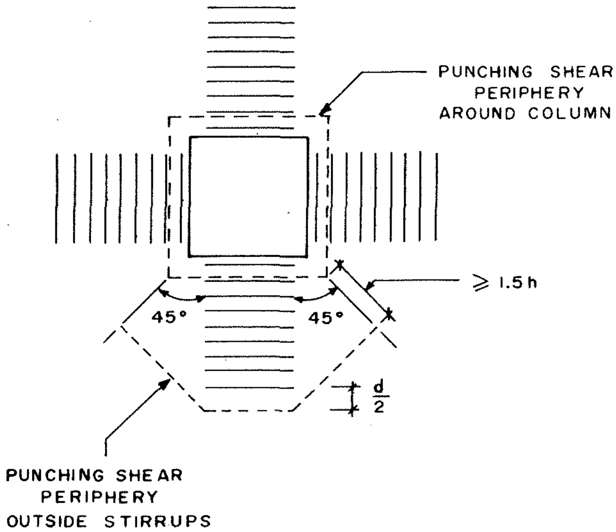

FIG. 10. Critical punching shear peripheries.

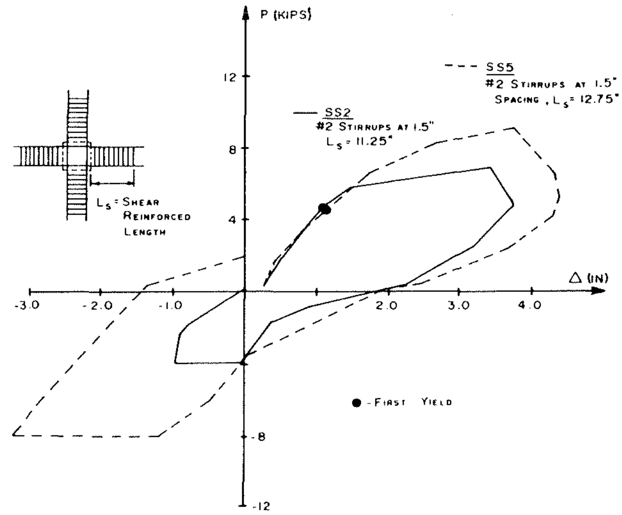

FIG. 11. Lateral load *versus* deflection envelopes for specimens SS2 and SS5 (1 kip = 453.6 kgf, 1 in. = 2.54 cm).

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 a definite limit to the amount of reinforcement that is effective when concentrated within the column region. 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 reinforcement 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

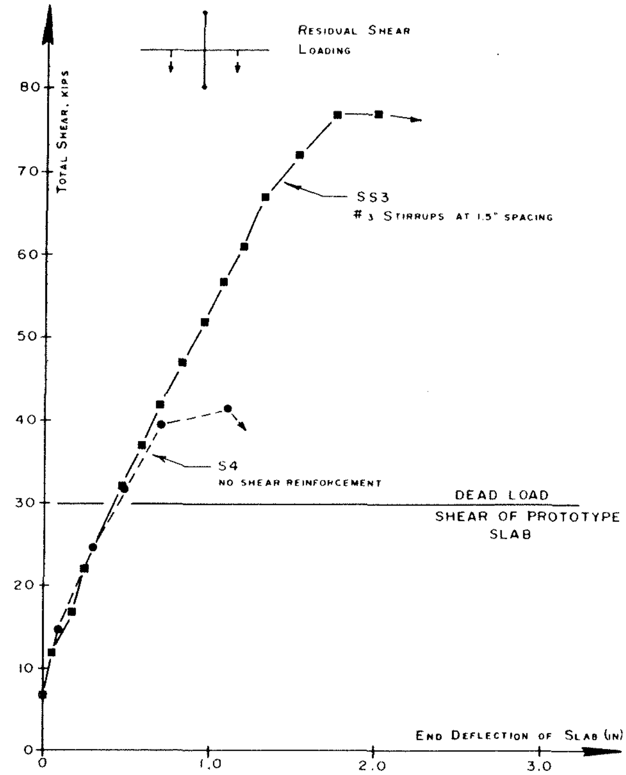

FIG. 12. Residual shear *versus* deflection behavior for specimens S4 and SS3 (1 kip = 453.6 kgf, 1 in. = 2.54 cm).

ratio for a section containing tensile reinforcement only (Hawkins 1975). 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 integral beam stirrup reinforcement is provided in the slab.

(2) The beneficial effects of providing properly designed and detailed integral beam stirrup 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 behavior 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 must be detailed so that: (a) they

are closed hoops with a longitudinal reinforcing bar in each corner, (b) they are anchored by 135° standard bends around one or more longitudinal bars, and (c) they extend far enough out from the column face into each column strip that the wide beam shear force V_u on the shear periphery shown in Fig. 10 does not result in a shear stress V_u/bd exceeding $2\sqrt{f_c'}$ ($0.53\sqrt{f_c'}$ if f_c' is expressed in kg/cm^2) and that perimeter does not approach closer than $1.5h$ to the column perimeter.

(4) The behavior 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 the ACI procedure (American Concrete Institute 1971) or the beam analogy (Hawkins 1975). For slabs with low reinforcement ratios (reinforcement ratios less than about 0.8%) subjected to monotonic loading the ACI procedure is nonconservative while the beam analogy correctly predicts measured strengths. For reversed cyclic loading both the ACI procedure and the beam analogy are slightly nonconservative for low reinforcement ratios.

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