HYSTERETIC BEHAVIOR OF CONCRETE SLAB TO COLUMN CONNECTIONS

by

Neil M. Hawkins[I], Denis Mitchell[II], and D. W. Symonds[III]

SYNOPSIS

Reversed cyclic loading tests are summarized for 26 specimens modeling connections in flat plate structures. Variables included the intensity of the reversing moments transferred to the column, extent and amount of flexural and stirrup reinforcement in the slab, and column proportions. Connections without shear reinforcement failed suddenly by punching. Connections with properly designed stirrups developed rotations eight times those for first yield of the slab reinforcement without significant loss in energy absorption with cycling. Recommendations are made for proportioning flexural and stirrup reinforcement and assessment of strength, stiffness and energy absorption of such connections.

INTRODUCTION

Punching failures at columns during recent earthquakes emphasize the wisdom of building ordinances prohibiting use of flat plate frames to resist seismic forces. However, if such failures can be prevented flat plate frames have high energy absorbing potentials. The test program reported here has defined the strength, stiffness and hysteretic characteristics of cyclically loaded plate-interior column connections, and shown that flat plate frames can be used to resist seismic forces provided the true stiffness of the column connections is recognized and adequate, properly detailed, stirrup reinforcement provided in the slab. Comprehensive details of the test results and their correlation with results of other investigators (1-3) are given in References 4-9.

The test specimens had the proportions shown in Fig. 1. They modeled to full scale the slab-column region of a flat plate structure designed for gravity loads according to ACI Code 318-71 and having 20 ft. square panels (4,6). For the type A, "moment-transfer", specimens there were two separate jacking systems allowing independent variations of "gravity" and "lateral" load effects. For the type B, "shear-transfer" only specimens, loads were applied in opposite directions at the load points and on the axis of the column. All columns were preloaded to the axial force likely for a connection two floors below the roof.

TEST PROGRAM

Tests were made on the five series of specimens shown in Table 1. None of the specimens in Series I contained stirrup reinforcement. Gravity loads equaled the likely dead load for the prototype panel. In four specimens, the top reinforcement was uniformly distributed and the ratio either 0.6, 0.9 or 1.3%. In the fifth specimen, 1.9% reinforcement was concentrated within lines twice the slab thickness either side of the column. Outside that region, the ratio was 0.6%. Series II specimens contained integral beam shear

I Professor of Civil Engineering, University of Washington, Seattle, WA
II Assistant Professor of Civil Engineering, McGill Univ., Montreal, Canada
III Research Engineer, University of Washington, Seattle, WA

reinforcement consisting of closed hoops encircling the reinforcing bars passing through the column. Those hoops terminated in standard 135° hooks around one bar. No. 2 stirrups were used in slabs with 0.9% reinforcement and No. 3 stirrups in slabs with 1.3% or concentrated reinforcement. Stirrups extended on 1.5 in. centers to varying distances from the column. In Series III, the gravity load was increased to the dead plus full live load likely for the prototype panel and specimens made identical to selected specimens from Series I and II. For two specimens in Series IV the column was 16 in. square and for four specimens, 8 by 19.5 in. with the long dimension in the direction of moment transfer for two of the specimens and transverse to that direction for the other two specimens. Within each pair the larger gravity load was used for one test and the smaller gravity load for the other. Type B specimens were used for Series V only. For one specimen, the closed hoops were replaced by single leg vertical stirrups terminating in standard 135° hooks. Otherwise, reinforcement details were similar to selected specimens from Series I and II. Uniformly distributed bottom reinforcement with a ratio equal to half the average top reinforcement ratio was used in all specimens.

Typical load histories are shown in Fig. 1. The history for type A specimens was evolved after several variations in Series I and II. The lateral load was increased until the reinforcement passing through the column yielded. The loading direction was reversed and the opposite end of the slab deflected downward by the same amount from the initial gravity load position. The slab ends were then displaced three times between those limiting deflections, the limits increased equally, and the pattern repeated. For type B specimens, the corner forces were cycled three times between constant load limits, those limits altered by 0.25 times the initial load and the pattern repeated. Loading was discontinued when the specimen's capacity dropped to less than 50% of the maximum or the jack stroke was exhausted.

RESULTS

Load History. The maximum capacity achieved in the tests and the rotation of the slab relative to the column at that capacity decreased as the severity of prior cyclic loadings increased. Before the slab reinforcement yielded, the decrease in capacity with cycling between constant deflection limits was small. After yielding, that decrease was more marked with 80% of the change occurring between the first and second cycle. After crushing of the concrete at the column face, the percentage decrease in capacity with cycling doubled.

Flexural Reinforcement Ratio and Distribution. Shown in Fig. 2 are plots of the shear force on one half of the specimen versus the rotation of the slab relative to the column at that face. When any significant moment was transferred to the column, the rate at which rotations developed increased rapidly once the reinforcement passing through the column yielded. The stiffness in the elastic range, the shear for yielding, the stiffness in the post-yield range, the maximum capacity and the deflection for that capacity increased as the reinforcement ratio in the column vicinity increased. Concentration of reinforcement in that region was markedly beneficial. Before the slab reinforcement yielded hysteresis loops were narrow, spindle-shaped and similar for all specimens. After first yielding, the loops became fatter. For a 12 in. square column the loops were spindle-shaped in the post-yield range only if the reinforcement ratio in the column vicinity was 0.9% or less. For greater reinforcement ratios, post-yield loops were increasingly S-shaped.

Shear Reinforcement. Shown in Fig. 3 are plots for specimens similar to the specimens of Fig. 2 except for the omission of shear reinforcement. Specimens without shear reinforcement failed in a brittle manner and for severe cyclic loadings the increase in capacity beyond first yielding of the reinforcement was small. With properly detailed shear reinforcement, the ductility, energy absorption and strength characteristics of the connection improved markedly and rotations for first crushing of the concrete were in excess of five times those for first yielding. Further, with such shear reinforcement, the imposition of rotations greater than those for crushing caused a loss in strength but not a decrease in energy absorption. With shear reinforcement there was a 10 to 20% increase in the stiffness of the connection and in the fattiness of the hysteresis loops for a given applied load.

Stiffness. Prior to first yielding of the slab reinforcement less than 10% of the slab's edge deflection was caused by column rotations. The actual contribution agreed well with that calculated assuming the column, cracked or uncracked, as appropriate for the combinations of axial load and moment acting on it. Thirty to 40% of the edge deflection resulted from bending of the slab between the column and the edge. The actual contribution agreed well with that calculated assuming the slab, cracked or uncracked as appropriate, acting as a beam with a width equal to that of the test specimen. A concentrated rotation at the column face accounted for the remaining portion of the edge deflection. Edge deflections were more than double those predicted using the equivalent frame method of ACI Code 318-71 and a cracked section for the slab. Further, the relative contributions of each factor did not change significantly for the inelastic range. Comparisons of the test results for Series IV with those for Series I, II and III and with finite element analyses showed that the magnitude of the maximum shear stress acting on the connection and the anchorage conditions for the reinforcement passing through the column dictated the concentrated rotations at the column face and the conditions under which the hysteresis loops became S-shaped. An increase in the column dimension in the direction of moment transfer improved bond conditions for the slab reinforcement, decreasing the concentrated rotation by about 20% and resulting in the hysteresis loops not becoming S-shaped until considerably higher shear stresses. An increase in the column side length from 12 to 16 in. reduced the concentrated rotation by about 40%, due to a decrease in the shear stress and an increase in the development length. For that column size and concentrated reinforcement pinching effects for the hysteresis loops were virtually eliminated. Stiffness variations for loading and unloading from various load levels were consistent with Takeda, Sozen and Nielsen's model (10).

Damping. Damping coefficients for the specimens were determined as indicated in Fig. 4a. Coefficients for loading to a new peak were typically 10 percent in the elastic range and 14 percent in the inelastic range. Those values decreased to about 8 and 12 percent respectively for the second cycle between the same deflection limits. Values increased slowly with decreasing slab reinforcement ratios, with better anchorage for the slab reinforcement within the column, with lower unit shear stresses on the connection for a given reinforcement stress and with increasing inelastic action.

RECOMMENDATIONS

Provision of Shear Reinforcement. Integral beam shear reinforcement is desirable for all flat plate connections likely to transmit seismic forces. For the connection shown in Fig. 4b the shear stress above which shear reinforcement is necessary depends on the characteristics of the loading. If the direction of the panel moments on lines AB and DC does not reverse, stirrups should be provided whenever the shear stress on any side length of the section efgh exceeds $3\sqrt{f_c'}$ (I). If the panel moment directions reverse the limiting stress should be decreased to $2\sqrt{f_c'}$ and if the reversals crack the slab through its depth the limiting stress should be $1.8\sqrt{f_c'}$. Stirrup reinforcement should be designed to carry all shear stresses in excess of $1.8\sqrt{f_c'}$ and not to yield under maximum load conditions. The spacing for the stirrups should not exceed d/3 and every longitudinal bar passing through the column should be located at the corner of a stirrup. Stirrups should terminate with standard 135° hooks around flexural bars. The reinforcement should extend far enough from the column that (1)the distance ed exceeds the slab thickness, h, and (2)the nominal shear stress on the section abcd does not exceed $2\sqrt{f_c'}$ for a slab not cracked through its full depth and $1.8\sqrt{f_c'}$ for a slab cracked through its depth. The shear force acting on abcd should be taken as the sum of the lateral load shear for the panel CD plus the gravity load shear flowing to the column through that section.

Proportioning of Connection and Flexural Reinforcement. In order to minimize pinching effects for the hysteresis loops the maximum shear stress should not exceed $6\sqrt{f_c'}$ and the column length in the direction of moment transfer should be greater than the anchorage length for the slab reinforcement. Consistent with the likely elastic moments there should be a concentration of slab reinforcement within lines three times the slab thickness either side of the column and gravity loadings should not stress that steel inelastically. If the anchorage length for the slab reinforcement is not less then half the column dimension in the direction of moment transfer, the sum of the ratios for the top and bottom slab reinforcement, considered effective for transfer of moment, should not exceed 0.75 times the balanced reinforcement ratio.

Ultimate Strength. The strength of a connection is best determined using the beam-analogy (11)(12). The ACI Code 318-71 procedure will overestimate strengths for sections with less than 0.9% reinforcement in the column region. Where the panel moment directions reverse during the seismic loading strengths should be reduced by the ratio of the cover over the bottom reinforcement to the effective depth to the top reinforcement.

Stiffness. Prior to yielding of the slab reinforcement the model shown in Fig. 4c correctly predicts the connection stiffness and the changes in that stiffness with slab reinforcement, gravity loadings, column shape and column size. The slab is assumed attached to the column by two cantilevering flexural elements F_1 and F_2 and two torsional elements T_1 and T_2. Elements F_1 and F_2 have the properties of the slab in their respective regions and the loading and unloading stiffnesses defined in Reference 10 and shown in Fig. 4(c). Elements T_1 and T_2 have stiffness $(GJ)_{cracked}$ as indicated on Fig. 4(c)(13). The flexural elements are loaded and unloaded from the preexisting conditions for gravity loading and the importance of the reinforcement in those elements not being in the yield range is obvious. As indicated on Fig. 4(c) compatability for points A, B, C and D, determines the twist ϕ of the torsional elements.

(I) The notation of this paper is that of ACI Code 318-71.

For gravity load stresses in the reinforcement equal to about half the yield stress, the effective cracked section in the column region for lateral loadings varies between about c_2+d for a square column through to c_2+2d for a column with a c_1 to c_2 ratio of three. This model is not appropriate for the post-yield range and more complex procedures are necessary to predict that response (14).

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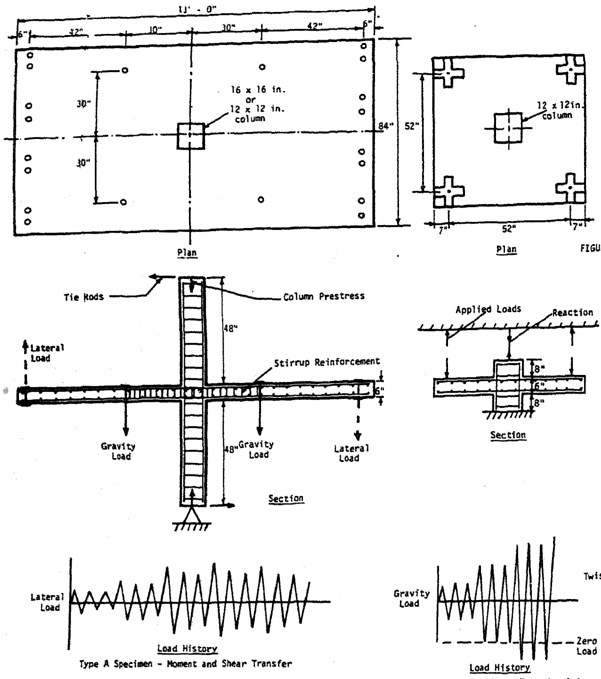
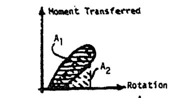


FIGURE 1 TEST SPECIMENS



Damping Coefficient = $\frac{1}{2\pi}\frac{A_1}{A_2}$

FIGURE 4(a) DAMPING COEFFICIENT

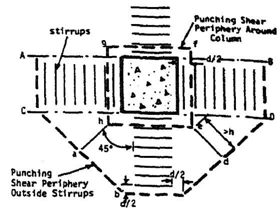
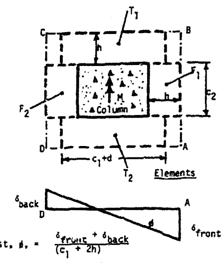


FIGURE 4(b) PUNCHING SHEAR PERIPHERIES

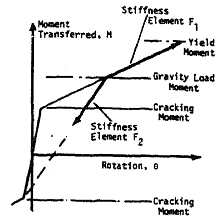
For T_1 and T_2, $(GJ)_{cracked} = \frac{2A_t\ (c_1 + 2d - d') + A_t s)\ 2.1(GJ)_{uncracked}}{100\ (c_1 + d)(h)(s)}$

where A_t = cross-sectional area of top bars of slab

and s = spacing of top bars of slab



Twist, ϕ, = $\frac{\delta_{front} + \delta_{back}}{(c_1 + 2h)}$



Flexural Stiffness

TABLE 1 - TEST PROGRAM

Series	Reference	Spec. Type	No. of Spec.	Concrete Strength Range -psi	Top Reinf. Ratios	Gravity* Load kips	Stirrup Reinf.	Variables
I	(4)	A	5	3200 -5050	0.6, 0.9, 1.3, 1.9-0.6	29 -34	None	reinf. ratio; load history
II	(5) (6)	A	5	3730 -4670	0.9, 1.3 1.9-0.6	28 -30	All	reinf. ratio; stirrup strength and extent; load history
III	(8)	A	5	3360 -4470	0.6, 0.9 1.9-0.6	53 -61	2spec	reinf. ratio; stirrup strength and extent.
IV	(9)	A	6	3360 -4290	1.9-0.6	29 -61	All	column size and shape; gravity load; stirrup strength and extent.
V	(7)	B	5	3200 -4500	0.9. 1.9-0.6	42	3spec	reinf. ratio; stirrup strength, extent and type

* includes self weight of test slab and loading apparatus

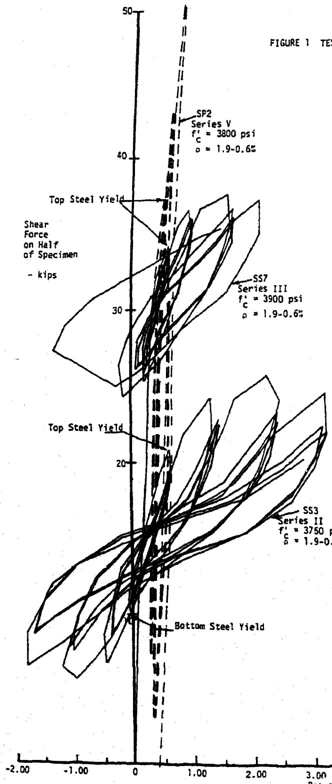


FIGURE 2. LOAD-ROTATION RELATIONSHIPS FOR SPECIMENS WITH INTEGRAL BEAM SHEAR REINFORCEMENT

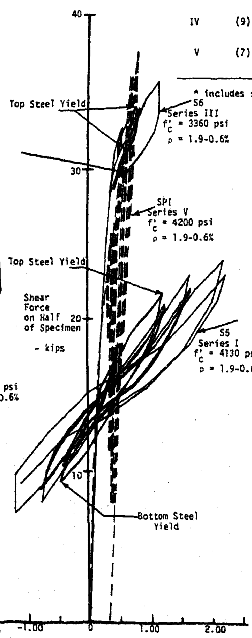


FIGURE 3. LOAD-ROTATION RELATIONSHIPS FOR SPECIMENS WITHOUT SHEAR REINFORCEMENT