SEISMIC RESISTANCE OF UNBONDED

POST-TENSIONED FLAT PLATE CONSTRUCTION

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SUMMARY

Tests to failure on six full-scale unbonded post-tensioned prestressed slab-column subassemblages subjected to loading in which reversing moments were transferred to the column are described. It is demonstrated that interior-column sub-assemblages will behave in a ductile manner if the provisions of ACI Code 318-77 are satisfied and that bundling of tendons through a column or over a lifting collar is an effective means of increasing the moment transfer strength and lateral load stiffness of such sub-assemblages.

INTRODUCTION

Two-way post-tensioned flat-plate structures are frequently the cheapest form of construction for parking garages, apartments and offices. There are two common forms for these structures: slabs cast monolithically with columns and lift slabs. For both forms, economics and speed of construction considerations dictate that the slabs be post-tensioned with unbonded tendons. There is a wide divergence of opinion on whether unbonded tendons should be allowed in prestressed structures in seismic zones. There is also a wide difference of opinion on whether flat-plate construction should be permitted, even as part of the gravity load carrying framework in moderate to active seismic zones. If such construction is permitted, the slab-column connections must be capable of transferring all required loads at the maximum deformations likely for those connections during a severe earthquake. The main objective of this study was to determine the factors dictating the strength and stiffness of prestressed concrete plate-column connections transferring moments.

TEST SPECIMENS

Proportions for the test specimens were selected after detailed examination of a typical two-way unbonded flat-plate structure having approximately 20-ft. spans and designed according to ACI Code 318-77 for a live load of 40 psf together with a superimposed dead load of 15 psf (1). Columns 16-in. square were needed for shear and a 5.5-in. thick slab prestressed biaxially to 150 psi for stiffness. The proportions, loading and prestressing for the test specimens were chosen so that the stress and deformation conditions for their slab-column connections would simulate closely those for the connections in the prototype structure under a base shear of 0.05 times the weight of the structure.

The test program is summarized in Table 1. Five interior column-slab sub-assemblages, including one lift-slab specimen and one exterior column-slab sub-assemblage, were tested. Plan and longitudinal section details for

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the typical interior column specimen 1 are shown in Fig. 1. For cast-in-place connections, the 16-in. square columns required for the prototype structure were reduced to 14 in. in the model in order that, according to ACI Code 318-77, shear rather than flexural effects would govern. Frequently, in order to make tendon placement easier, banded construction as shown for the transverse direction in Fig. 1 is used. Since the structural engineering profession has posed many questions concerning the behavior and safety of banded construction, the effects of banding were investigated in this study. In specimens 1 and 6, tendons were banded transverse to the direction of lateral loading while in specimen 3, tendons were banded in the direction of lateral loading. The remaining specimens had distributed tendons. As indicated in Fig. 1, bundled tendons were anchored off against steel spreader beams whose reaction points ensured the desired level of axial prestress in the slab in the column region. Teflon bearing pads were placed beneath the steel spreader beams in order that the stiffness of those beams had minimal effect on the moment transfer characteristics of the sub-assemblages.

All specimens contained bonded grade 60 reinforcement with the amount within lines one and one-half times the slab thickness either side of the column being 10 percent less than that required by ACI Code 318-77. Outside the central region, additional bonded bars provided a flexural capacity for the slab equal to the negative moment flexural capacity center-to-center of panels for the prototype structure. Bonded bottom reinforcement consisted of two No. 4 bars in each direction through the column and another two No. 4 bars outside the column. In the plan view in Fig. 1, bonded bars are indicated by broken lines and tendons by solid lines. The general form of the lifting collar for specimen 6 is shown in Fig. 3. A 10 WF 72 steel section was used for the column and a 4-in. 7.2 lb/ft. channel for the lifting collar. The edge column specimen was of Type C (3). The tendons were draped only to mid-height of the slab at the edge containing the column.

The shear force acting on the connection in the prototype structure was simulated by jack forces applied at points A in Fig. 1. Moment transfer was caused by other jack forces applied at points B with the directions of those forces reversed for opposite edges of the specimens. The jack forces applied at A and B are termed the gravity and lateral loads respectively. The columns protruded 4 ft. above and below the slab. They were held in place at their tops by tie rods and pinned at their bases. For all interior column sub-assemblages except 5, the line ABCDEFJ in Fig. 2 indicates the overall loading history. Points A, D, etc., correspond to the loading conditions for the prototype structure shown in the Table in Fig. 2. To obtain some information on likely seismic effects, the forces transferred to the column were cycled three times over the ranges indicated by broken lines ED' and EE'. Reversals between E and D' yielded information on vertical acceleration effects while reversals between E and E' yielded information on horizontal acceleration effects. For specimen 5, the shear was increased until at A it equaled the dead-load shear, center to center of panels for the prototype structure. Then, as indicated by the broken line AP, the moment transferred to the column was increased until failure occurred. Information on seismic effects was again obtained by reversing the moments three times over the ranges JJ' and KK'. In practice, the load history for a lift-slab connection would differ from that for a cast-in-place connection since the self-weight of the slab would not cause moment transfer. However, to simplify comparisons, the load history for the lift-slab specimen was made the same as that for specimen 1 with essentially the same tendon layout.

TEST RESULTS

Lateral load-edge deflection relationships for the four high shear loading interior column sub-assemblages are shown in Fig. 4. The initial step function in the responses results from the edge deflection increasing when the gravity loads were increased without increase in the lateral loads. For the three cast-in-place connections, large deflections developed prior to failure. However, the lateral load capacity was still increasing rapidly with increasing deflections when punching failures occurred for all three specimens. Only the lift-slab specimen exhibited marked ductility prior to a failure caused by crushing of the concrete beneath one of the reaction blocks for the steel spreader beam. For the initial lateral loading, the greatest stiffness was shown by specimen 3 which had tendons bundled through the column in the direction of moment transfer. However, at conditions approaching failure, specimen 3 showed larger increases in deflection with increasing load than specimen 1. In that range, most of the increase in moment transfer resistance is provided by torsion in the slab sections framing into the side faces of the column (3). For specimen 1, with tendons bundled through those side faces, the increase in resistance for a given twist would be larger than for specimen 3 with distributed tendons in that direction.

The energy dissipation with lateral load cycling was greatest for specimen 4 and least for specimen 3. For all three cast-in-place connections, the width of the hysteresis loops decreased rapidly with cycling. The non-yielding post-tensioning tendons caused elastic recovery effects in spite of yielding of the bonded top bars passing through the column. To obtain reasonable hysteretic damping in seismically loaded prestressed concrete structures, there must be a careful balancing of the amounts of prestress and reinforcing bars in the hinging region (4). For this limited test series, the best balance was provided in specimen 4 with distributed tendons for both directions of the slab. There was no decrease in energy dissipation with cycling for the lift-slab specimen 6 because that dissipation was through yielding at the connection between the steel collar and the steel column. For all four specimens, there was little energy dissipation with cycling of the gravity loads.

The solid curve in Fig. 5 is the lateral load-edge deflection relationship for specimen 5. The broken curves are the lateral load-edge deflection envelopes for two comparable reinforced concrete specimens, one without shear reinforcement, S1, and one with integral beam-shear reinforcement, SS2 (5, 6). The negative moment flexural capacities on a line extending across the width of the slabs at the column face were comparable for all three specimens and the connections for all three specimens transferred similar gravity load-shear forces. However, the columns were 12-in. square for the reinforced concrete specimens. The reinforced concrete slab without stirrups, S1, failed abruptly in punching shear at a lateral load of 5.5 kips. The slab with stirrups, SS2, had a lateral load capacity of 8.5 kips and developed large deflections prior to the capacity decreasing with cycling and increasing deflections for deflections in excess of 4 in. Even after significant reversed cyclic loading, the post-tensioned slab still developed an ultimate ductility for uni-directional loading comparing favorably with the reinforced concrete slab with integral beam stirrups. For the prestressed slab cracking at the column face during the first loading to 2.0 kips resulted in a permanent edge deflection for zero lateral load. For the second and third cycles between lateral loads of 2.0 kips and for the fourth, fifth and sixth cycles between lateral loads of 3 kips, the hysteresis loops remained stable and spindle shaped. Although there was little energy dissipation, there was also no degeneration in stiffness with cycling.

For the slab-exterior column sub-assemblage 2, a loading history similar to that shown in the table of Fig. 2 was used. For that specimen, both gravity and lateral loadings caused moment transfer but at different ratios for equal increases in shear. Torsional cracking developed at the slab edge modeling the discontinuous edge of the prototype structure during the first cycle to load stage E, Fig. 2. Those cracks opened significantly during the subsequent cycling between the load stages corresponding to E and E' and had widths of 0.2 in. after three cycles. The capacity of the specimen for increases in lateral loading beyond stage E was small. It was obvious that hairpin stirrups of the type described in Reference 3 inserted perpendicular to the edge would have considerably improved the response of specimen 2.

These results show that prestressing can provide an excellent means for tying a slab together and ensuring ductile behavior for high intensity wind loadings. The stable reversed cyclic loading characteristics of these slab-interior column sub-assemblages also suggests that with proper detailing, post-tensioned unbonded slabs will perform satisfactorily under seismic loadings. However, reversed cyclic loading tests of higher intensity than those reported here are necessary to validate that potential and develop the understanding necessary to ensure ductile behavior for intense seismic loading.

LATERAL LOAD STIFFNESS

The results for the 0 to 2 kip lateral load range of specimens 1, 3, 4 and 5 provide information relevant to evaluations of the lateral load stiffness of prestressed concrete flat plate structures. For the 0 to 1.0 kip lateral load range, the stiffnesses varied with the tendon layouts ranging from the highest value for specimen 3 with tendons bundled in the lateral load direction down to the least value for specimen 4. For the 1.0 to 2.0 kip lateral load range, all slabs were visibly cracked in the column region and the lateral stiffnesses for all specimens were the same.

Because of the manner in which prestressed slabs are designed, it is difficult to predict when cracking will occur in a prototype structure. The lateral load response of these specimens was obviously sensitive to cracking and stiffnesses dropped even before cracking was detected. Further investigations of lateral load stiffness are needed. Analysis of these results showed that for high levels of prestress in the direction of moment transfer, the slab can be taken as uncracked and the lateral load stiffness assessed using ACI Code 318-77, Section 13.7 provisions. If, however, the level of prestress is low, the slab should be taken as cracked in bending and the torsional stiffness as one-tenth that predicted by Eq. (13-7) of ACI 318-77.

MOMENT TRANSFER STRENGTH

For an interior column connection, Section 11.12.2 of ACI Code 318-77 requires that shear stresses on a critical section located d/2 from the column perimeter be calculated from the formula:

$$v_u = \frac{V_u}{A_c} + \frac{\gamma_v M_u c}{J_c} \qquad (1)$$

where $\gamma_v M_u$ is the fraction of the total moment M_u transferred by shear across the critical section, A_c and J_c are the area and polar moment of inertia of the critical section, and M_u is the moment acting about the centroid of the critical section and to be transferred to the column. If the stress v_u exceeds some critical value v_c, a shear failure is predicted. Section 13.3.4 of ACI 318-77 requires that reinforcement be provided within lines one and one-half times the slab thickness either side of the column to transfer the fraction of the moment $(1-\gamma_v)\ M_u$ not transferred by shear. If there is insufficient reinforcement, a cut-off on the moment transfer capacity results. ACI-ASCE Committee 423 (7) has recommended that Eq. (11-13) of ACI 318-77 be used to determine the shear strength of connections transferring shear only. For slabs Eq. (11-13) becomes:

$$v_c = 3.5\sqrt{f'_c} + 0.3\ f_{pc} + V_{p/A_c} \geq 4\sqrt{f'_c} \qquad (2)$$

where f_{pc} is the axial prestress in the slab and V_p is the sum of the vertical components of all prestressing tendons crossing the critical section for shear.

The correlation between the measured capacities and those predicted by the application of the foregoing provisions for slab-interior column connections is apparent from Figs. 2 and 3. The line UV on the right of Fig. 2 indicates the moment cut-off capacity for specimens 1, 4 and 5. The reinforcement effective for moment transfer was taken as 4 No. 4 top bars, 4 No. 4 bottom bars, and two tendons stressed to 25 kips each. For specimen 3, line UV would be further to the right due to the larger number of longitudinal tendons. The sloping line, WX, on the right of Fig. 2 indicates the theoretical relationship for the negative moment flexural capacity of the slab of specimens 1, 4 and 5. The measured moment capacities for failure for specimens 1, 3, 4 and 5 are indicated by the vertical lines s_1t_1, s_3t_3, s_4t_4, and s_5t_5 crossing the load history lines. The adjacent sloping lines y_1z_1, y_3z_3, y_4z_4, y_5z_5 indicate the respective shear capacities when the shear stress calculated from Eq. (1) is limited to the value obtained from Eq. (2) with f_{pc} taken as the axial prestress in the direction of moment transfer. All specimens failed by punching and the measured capacities were slightly greater than the predicted, shear-stress limited, moment transfer capacities for all specimens except 4. Specimen 4 had a capacity only two percent less than the predicted capacity. In Fig. 3, the notation for each line is the same as that for Fig. 2. As recommended in Reference 8, the shear stress caused by moment transfer (second term of Eq. (1)), was evaluated using the critical section ABCD shown on the insert in Fig. 3. The shear stress caused by shear transfer (first term of Eq. (1)), was evaluated using the critical section EFGH. The lift slab specimen failed due to excessive deflections. That behavior was in close agreement with the predicted behavior.

Moment transfer causes upward shear stresses at the back face of a slab-column connection. For reinforced concrete connections, cracking occurs when the net stress due to shear and moment transfer exceeds $0.3\sqrt{f'_c}$ upwards. In these tests on prestressed concrete slab-interior column connections, that condition was seldom reached. However, cracking developed at the back faces and significant tensile stresses were measured in the bottom reinforcement passing through those faces well in advance of failure. No simple criterion, such as that for reinforced concrete, could be established for conditions for cracking. For the high moment specimen 5, the reinforced concrete criterion was satisfactory. However, for the high shear specimens 1, 3 and 4, cracking apparently occurred as soon as the top steel yielded provided the $\gamma_v M_u c/J_c$ term in Eq. (1) exceeded $0.7\sqrt{f'_c}$. Further investigations of bottom reinforcement requirements are highly desirable.

CONCLUSIONS

Based on the prestressed concrete test results reported here and their correlation with available data for similar reinforced concrete subassemblages, it is concluded that the moment transfer capacity of prestressed concrete slab to interior column connections can be evaluated using the procedures of Sections 11.12.2 and 13.3.4 of ACI Code 318-77. For dense aggregate concretes, the shear stress v_u should not exceed the value of v_c calculated from Eq. (11-13) of the Code expressed in the form of Eq. 2 of this paper. All reinforcement, bonded and unbonded, within lines one and one-half times the slab thickness either side of the column are effective for transferring the portion of the moment not transferred by shear. Tendons bundled through the column or over the lifting collar are an effective means of increasing the moment transfer strength of such connections.

ACKNOWLEDGMENTS

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TABLE 1 PROPERTIES OF TEST SPECIMENS

Slab No.	Type	Concrete Strength* f'_c (psi)	Prestress				Loading	Shear kips	Moment ft-kip
			Transverse		Longitudinal				
			Type	fpc psi	Type	fpc psi			
1	Interior	3850 (3920)	Bundled	160	Distributed	160	Proportionate	66.8	51.5
2	Exterior	4200 (4100)	Two through Column	160	Two through Column	160	Proportionate	30.8	37.0
3	Interior	3650 (3900)	Distributed	160	Bundled	275	Proportionate	67.8	65.4
4	Interior	3800 (4140)	Distributed	160	Distributed	160	Proportionate	69.9	29.9
5	Interior	3600 (4200)	Distributed	160	Distributed	160	Shear constant at dead load, moment increased to failure	25.2	99.0
6	Interior Lift Slab	3000	Bundled	160	Distributed	160	Proportionate	64.0	78.0

* Values in parentheses are top column strengths

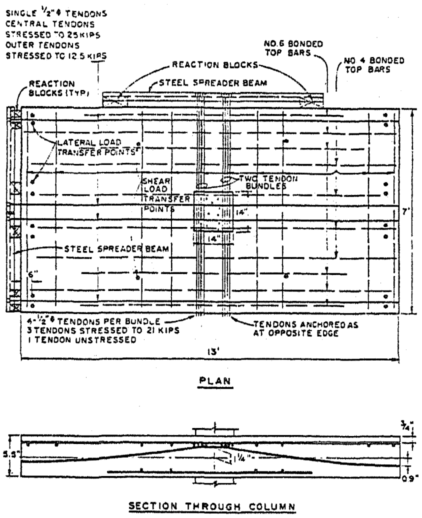


FIG. 1 PROPERTIES OF TYPICAL SPECIMEN 1

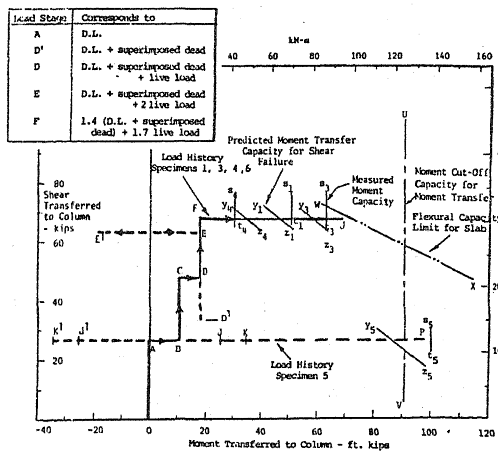


FIG. 2 LOAD HISTORY AND MEASURED AND COMPUTED STRENGTHS FOR INTERIOR COLUMN SPECIMEN

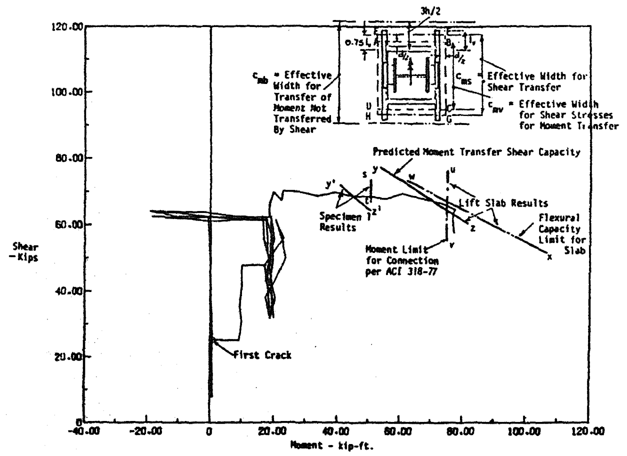
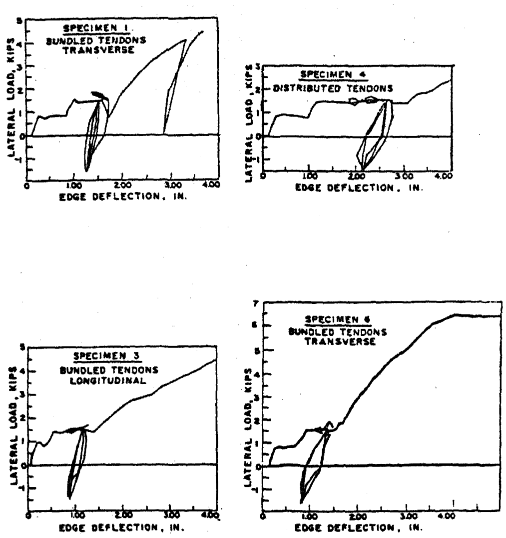


FIG. 3 LOAD HISTORY AND MEASURED AND COMPUTED STRENGTHS FOR LIFT SLAB SPECIMEN



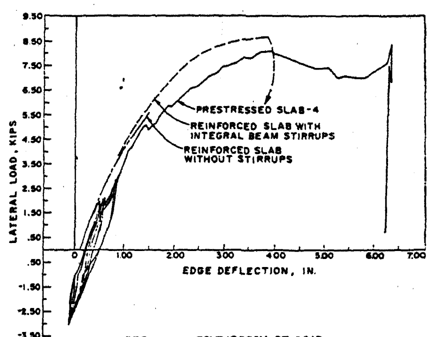


FIG. 5 COMPARISON OF LOAD-DEFLECTION DIAGRAMS FOR REINFORCED AND PRESTRESSED CONCRETE SUBASSEMBLAGES

FIG. 4 LOAD-DEFLECTION DIAGRAMS FOR INTERIOR COLUMN SPECIMENS