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REHABILITATION OF EARTHQUAKE-DAMAGED REINFORCED CONCRETE FLAT-PLATE SLAB-COLUMN CONNECTIONS FOR TWO-WAY SHEAR

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ABSTRACT

Experimental research on 2/3-scale slab-column connections was conducted to quantify the effects of earthquake-damage and low reinforcement ratios on the punching shear strength, and to study the efficiency of various rehabilitation techniques. Test results showed that connections with about 0.5 % top reinforcement ratio within the (c+3h) region, which is typical in the older flat-plate structures, had about two-third of the two-way shear capacity estimated using ACI 318-05 expressions. Installing external carbon fiber reinforced polymer (CFRP) stirrups and applying well-anchored CFRP sheets on the tension surface of the slab were effective in increasing the punching shear strength of the earthquake-damaged connections.

Introduction

A flat plate structural system provides more clear space for given story heights. However, the system is also prone to brittle shear failures at slab-column connections. For that reason, connections with insufficient two-way shear strength may need to be rehabilitated. Rehabilitation can be a cost-effective alternative to replacement.

Typical flat-plate structures built in 1960s or earlier do not have a concentration of the slab top steel in the column strip of the connection as required by the current building code (ACI 318-05). Structural drawings of several flat-plate structures located in San Francisco, show that those structures have about 0.5% flexural reinforcement ratio in the column strip and no shear reinforcement. Since low percentage of longitudinal reinforcement in the column strip results in low two-way shear strength (Marzouk and Hussein 1991), the rehabilitation of the earthquake-damaged connections in such structures usually involves flexural and shear strengthening.

Section 11.12.2.1 of ACI 318-05 states that the nominal punching shear strength V_c of a slab-column interior connection with a square column is the lesser of Eqs. 1 and 2.

$$V_c = \left(40 \times d/b_o + 2\right) \times \sqrt{f_c^{'}} \times b_o \times d \tag{1}$$

$$V_c = 4 \times \sqrt{f_c^{'}} \times b_o \times d \tag{2}$$

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where d is the average depth of slab reinforcement, b_o is the critical shear perimeter located at $d/2$ away from the edge of the column or from the outer most of the slab shear reinforcement, and f_c' is the concrete compressive strength.

Literature Review

In general, strengthening slab-column connections involves installing external shear reinforcement and/or collars to increase the critical shear perimeter. Several alternatives to increase shear capacity at the critical section include (i) steel bars grouted into 45-degree inclined drilled holes (Hassanzadeh and Sundqvist 1998), (ii) bolts to act as shear reinforcement (El-Salakawy et al. 2003), and (iii) carbon fiber reinforced polymers (CFRP) stirrups (Binici 2003). The shear perimeter has been increased by (i) installing column capital using reinforced shotcrete, (ii) attaching steel collars (Hassanzadeh and Sundqvist 1998), and (iii) sandwiching the slab between steel plates connected by through bolts (Ebead and Marzouk 2002). The flexural strength of a connection has been increased by applying CFRP on the slab surface thereby increasing the shear strength of the connection (Harajli and Soudki 2003, Tan 2000). The efficiency of CFRP is highly dependent on the ability to prevent an early delamination (Ebead and Marzouk 2004).

Test Program, Test Specimens, Test Setup, and Material Properties

In order to quantify the effects of earthquake damage and low reinforcement ratio on the punching shear strength, and to study the efficiency of various repair techniques, six two-third-scale specimens representing interior slab-column connections were tested (Table 1). The prototype structure for all specimens, except SP-Control 3, is an office building designed using ACI 318-1963 and 50 psf live load. The prototype structure was assumed to have 21-feet wide bays, 24-inch square columns, 9-inch slab thickness (based on the minimum slab thickness requirement of 8.4 inches), 0.5% slab top reinforcement in the column strip and 0.25% reinforcement elsewhere. No shear reinforcement was used in the slab. SP-Control 3, whose top reinforcement ratio is about the same as that in the flat-plate structures designed using ACI 318-05, served as a basis for comparing the strengthened specimens.

Grade 60 deformed reinforcement satisfying ASTM A 706-04 requirements and 4000-psi concrete were used in the experimental program. The properties of materials used in this study are summarized in Table 2. The details of slab reinforcement are shown in Fig. 1. The top reinforcement consists of US #4 and #3 bars within the column strip and the middle strip, respectively. US #3 bars were used as bottom reinforcement. The top and bottom reinforcement in the lateral loading direction had a clear cover of 0.5 inches. The average depth of slab reinforcement, d, was 5 inches.

For lateral loading, three fully-reversed displacement cycles were applied at each lateral drift level. The column axial load was controlled so that the shear force on the critical shear perimeter was maintained at $0.23\left(4\sqrt{f_c'}b_o d\right)$ and represented dead load plus 25% of live load on the prototype structure.

Table 1. Test program, specimens, and results.

Specimen	Test program	f_c' (cylinder) (psi)	% slab top steel within (c+3h)*	Punching load, V (kip)	ACI critical shear perimeter, b_o (in)	Shear stress at critical perimeter, $v = V / (b_o d)$
SP-Control 1	Lateral loading up to failure	3710	0.5	-	-	-
SP-Control 2	Punching shear loading only	4550	0.5	69.9	84	2.47 $\sqrt{(f_c')}$
SP-Control 3	Punching shear loading only	4070	1.0	90.2	84	3.37 $\sqrt{(f_c')}$
SP-1	Lateral loading up to 1.25%, then punched	4860	0.5	72.7	84	2.48 $\sqrt{(f_c')}$
SP-2	Lateral loading up to 1.25%, rehabilitated by external CFRP stirrups, then punched	4930	0.5	86.5	84** / 135***	2.93 $\sqrt{(f_c')}$ / 1.83 $\sqrt{(f_c')}$
SP-3	Lateral loading up to 1.25%, rehabilitated by CFRP sheet, then punched	4630	0.5	97.5	84	3.41 $\sqrt{(f_c')}$

** c=16" (column dimension of the specimen), h=6" (slab thickness of the specimen)*

*** b_o=84" was calculated for the critical perimeter d/2 away from the column face (d=5").*

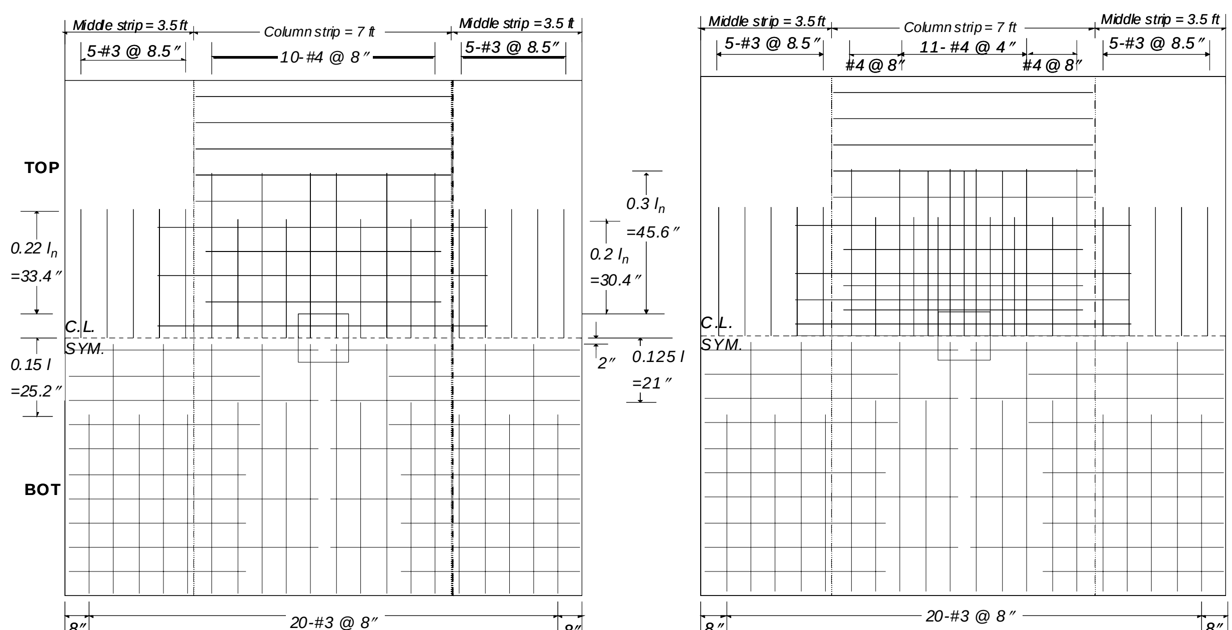
**** b_o=135" was calculated for the critical perimeter d/2 away from the outermost CFRP stirrup (d=5").*

Table 2. Material properties.

Material	Properties	Value
Concrete	Compressive strength	4 ksi (specified) ; 3.7 ksi - 4.9 ksi* (actual)
US #3 bar	Yield strength	GR 60 (specified) ; 63 ksi (actual average)
US #4 bar		GR 60 (specified) ; 66 ksi (actual average)
CFRP**	Elastic modulus	10500 ksi (specified)
	Ultimate tensile stress / strain	127 ksi / 0.012 (specified)

* See Table 1

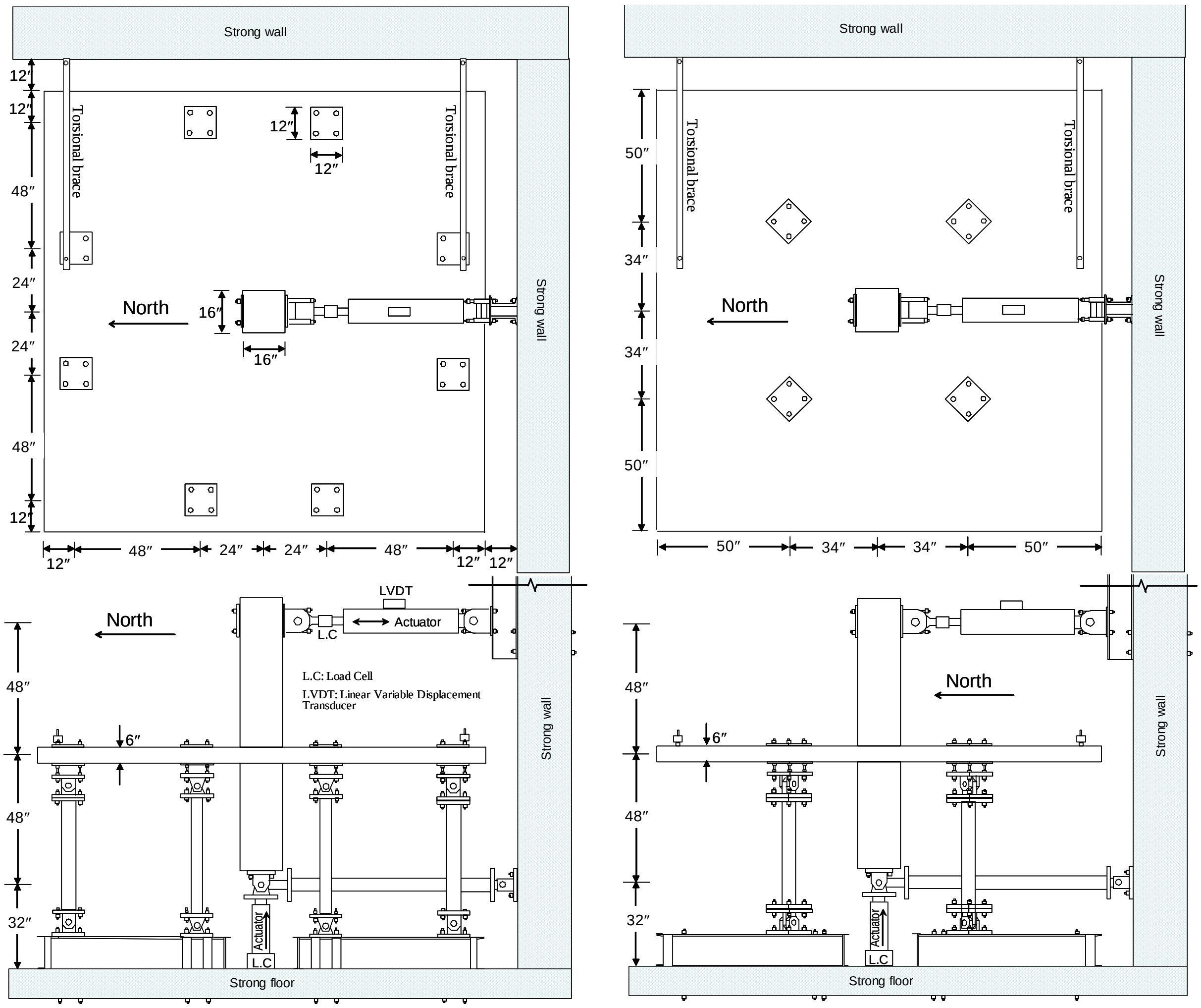
** 0.04-inch thick, unidirectional CFRP with aramid cross fiber



0.5% slab top steel within (c+3h) region   1.0% slab top steel within (c+3h) region

Figure 1. Slab reinforcement.

Figures 2a and 2b show the test setup used to test the specimens under simulated seismic loads (North-South direction) and punching shear loads, respectively. The locations of the vertical struts used in lateral loading and punching shear tests were different. The positions of the struts shown in Figure 2 were selected to reflect results of finite element analyses conducted on the prototype structure subjected to lateral and gravity loads.



(a) Lateral loading (b) Punching shear test

Figure 2. Test setup.

Loading Protocol and Rehabilitation Methods

SP-Control 1

Figure 3 shows the lateral load versus drift response of SP-Control 1. Before the application of lateral loads, a vertical load of 26.4 kip ($0.92\sqrt{f_c^{'}}b_o d$) was applied to the column. While maintaining the vertical load, the specimen was subjected to lateral displacement excursions shown in Fig. 3. The specimen failed in punching shear at 2% drift.

SP-1, SP-2, and SP-3

Since the maximum lateral load was reached at 1.5% drift, specimens SP-1, SP-2, and SP-3 were subjected to the displacement protocol shown in Fig.3 up to 1.25% drift and then were subjected to gravity load only to produce failure. In this way, considerable damage due to simulated seismic loads was induced but the connection did not fail.

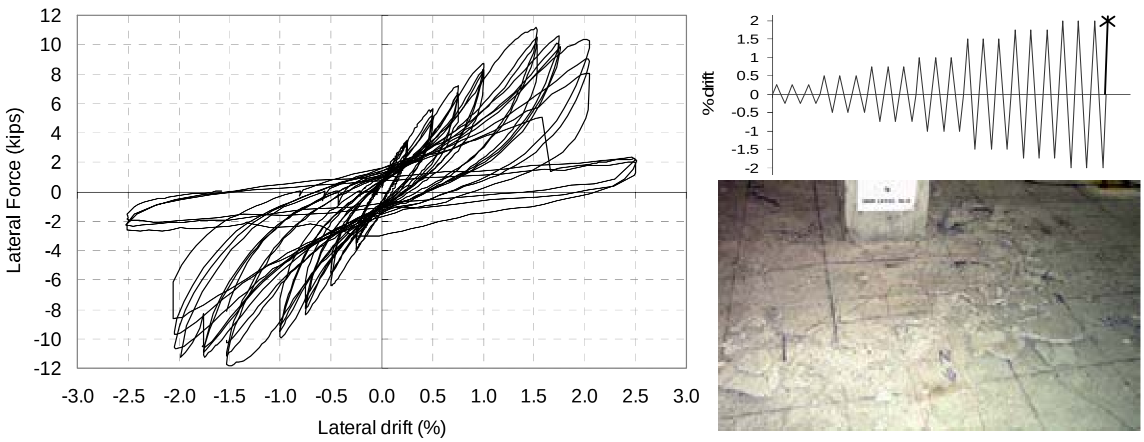


Figure 3. Lateral load versus drift and failure surface of SP-Control 1.

The boundary conditions for the gravity load test were changed by moving the struts into the position shown in Fig. 2(b). SP-1 was loaded to failure under gradually increasing gravity loads. SP-2 and SP-3 were rehabilitated before they were subjected to gravity load. In order to mimic the rehabilitation process in a flat-plate building that is unshored, the column axial load producing a shear force of $0.23V_c$ on the critical shear perimeter was maintained during rehabilitation process.

SP-2 was repaired by installing external CFRP stirrups around the column (Fig. 4). This rehabilitation technique was studied by Binici (2003). Fig. 5 shows the preparation process, which involved (i) locating the slab reinforcement using non destructive testing, (ii) drilling 3/4–inch holes, (iii) grinding the slab surface before CFRP stirrups application, and (iv) chamfering the edge of the hole to eliminate sharp edges. Three-quarter-inch wide CFRP strips were cut from a roll of CFRP fabric. The first row of CFRP stirrups were located as close to the column face as possible (d/4 away from column face) in order to intercept the shear crack. The other rows of CFRP stirrups were spaced at about d/2. After being impregnated with epoxy and passed through a saturator to remove excessive epoxy, the CFRP strips were stitched through the holes and wrapped to form closed stirrups. The CFRP strips were stitched once or twice through each hole, as shown in Fig. 4. CFRP overlaps of 6 inches the top slab surface were based on the test results reported by Binici (2003). After the completion of CFRP stirrup placement, the bottom of the vertical holes were plugged and the holes were filled with epoxy.

SP-3 was repaired by installing CFRP sheets around the column (Fig. 6). CFRP sheets were fastened to the slab tension face to increase the flexural capacity of the slab that had only 0.5% reinforcement ratio within (c+3h) region. With such a low reinforcement ratio, flexural failure is more likely to govern over the two-way shear failure (Marzouk and Hussein 1991). A 1.0% reinforcement ratio is considered to be typical in a slab-column connection designed using current codes. The amount of CFRP reinforcement was selected to produce the same flexural capacity within (c+3h) region as that of a connection with 1.0% steel.

The installation of CFRP sheets involved drilling four holes (¾-inch diameter and 4½-inch deep) at each corner of the column so that CFRP anchors could be inserted. The edges of the holes were chamfered and the concrete surface was ground smooth and cleaned. The CFRP installation is shown in Fig. 6. (i) Epoxy was poured into cracks and holes, (ii) concrete surface was coated with the epoxy, (iii) an impregnated CFRP sheet was inserted into a saturator to remove excessive epoxy, (iv) the sheet was placed on the concrete surface and a paint roller was used to remove air voids below the sheet, (v) CFRP anchors were inserted into the holes, and (vi) the protruding ends of the anchors were splayed over the CFRP sheet.

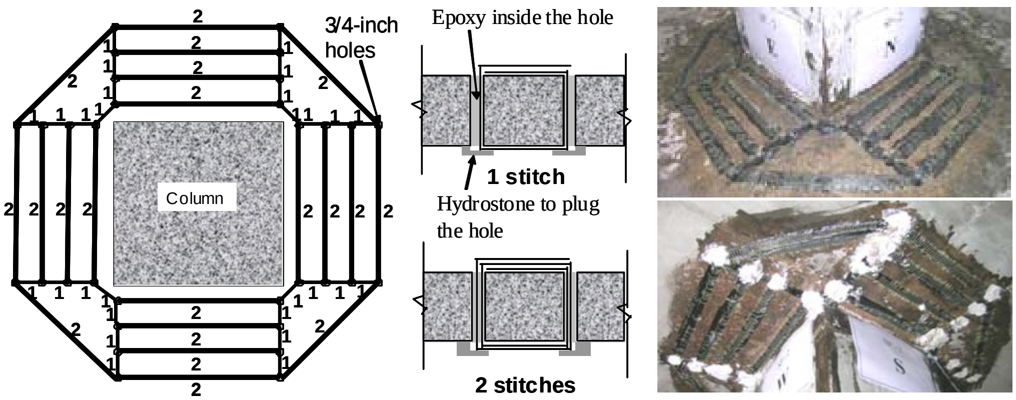


Figure 4. Externally installed CFRP stirrups for SP-2.

Figure 5. Hole and surface preparation.

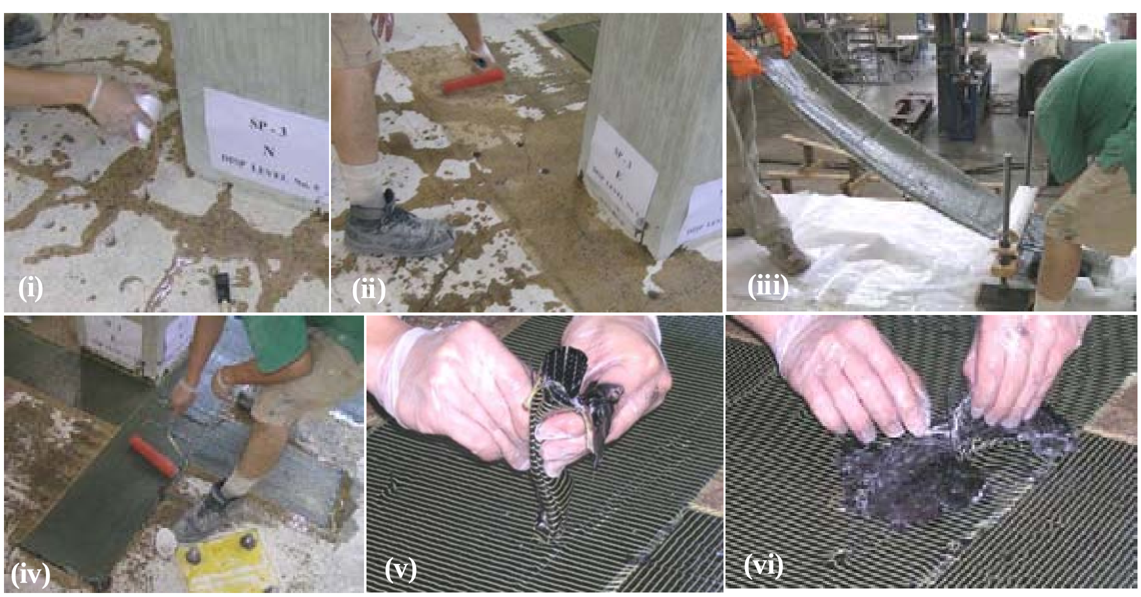


Figure 6. Installation of CFRP sheets and CFRP anchors in SP-3.

SP-Control 2 and SP-Control 3

SP-Control 2 and SP-Control 3 were loaded to failure under gravity loads to examine the two-way shear strength of connections undamaged by lateral loads.

Test Results and Discussions

In Fig. 7, gravity load versus vertical displacement (at the column) curves are shown. In Table 1, the gravity load capacity and the shear stress at the critical shear perimeter v_c are summarized for all specimens. For the slab column connections tested in this study (all specimens except SP-2), Eq. 2 governs the design. Table 1 shows that all of the specimens tested in this study failed at shear stress levels that were lower than $4\sqrt{f_c'}$, the value used in most designs. The critical shear perimeter of SP-2, which is at $d/2$ away from the outermost CFRP stirrup, is larger than that of the other specimens. Thus, Eq. 1 governs and gives v_c of $3.48\sqrt{f_c'}$ for SP-2. The measured v_c of SP-2 was $1.83\sqrt{f_c'}$.

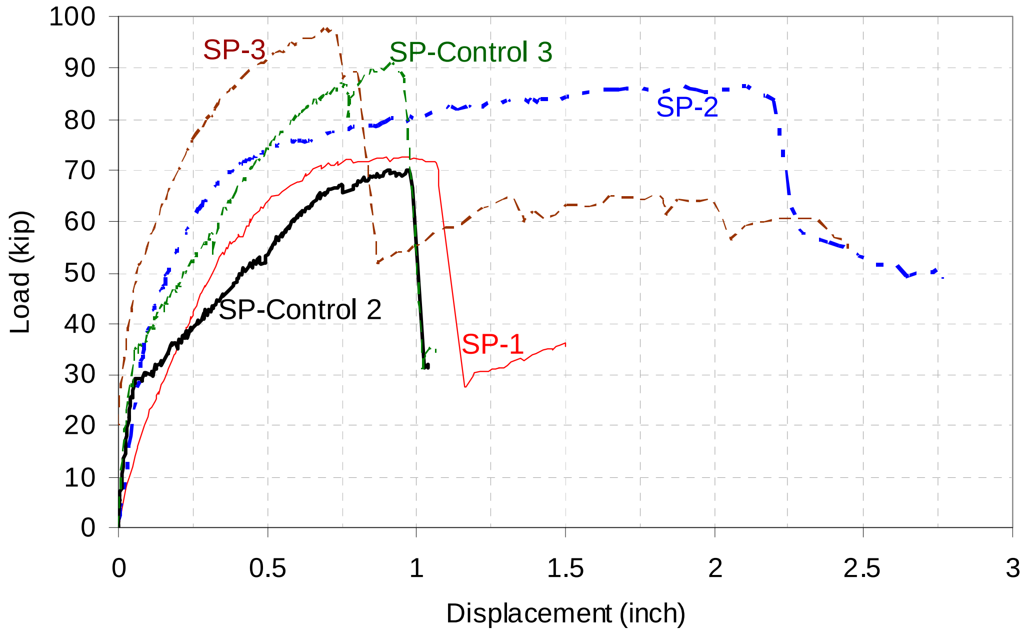


Figure 7. Load-displacement curves for punching shear tests.

The Effect of Longitudinal Reinforcement Ratio

The behavior of SP-Control 2 and SP-Control 3 can be used to evaluate the effect of longitudinal reinforcement. Specimens SP-Control 2 and SP-Control 3 contained 0.5% and 1.0% top flexural reinforcement within the critical (c+3h) region. At failure, v_c of specimens SP-Control 2 and SP-Control 3 were $2.47\sqrt{f_c'}$ and $3.37\sqrt{f_c'}$, respectively. It is interesting to note that SP-Control 2 represents the slab-column connection of the prototype structure. Even with a somewhat high reinforcement ratio, the two-way shear strength of SP-Control 3 was less than $4\sqrt{f_c'}$ recommended by ACI 318-05. This observation is consistent with the test results of a 45-foot square flat-plate structure (Guralnick and LaFraugh 1963).

In order to develop a feasible technique to alleviate the adverse effects of low amounts of flexural steel within the (c+3h) region, SP-3 was tested. This test specimen contained 0.5% reinforcement ratio within the (c+3h) region. In order to add more flexural reinforcement, 12-inch wide CFRP sheets were installed on the top face of the slab and anchored by CFRP fan anchors (Fig. 6). As can be seen in Fig. 7, the installation of well-anchored CFRP sheets on the tension side of the slab improved the punching shear strength of the slab-column connection. Left unrepaired, the capacity of SP-3 would have resembled that of SP-Control 2. SP-3 can be considered as the repaired version of SP-Control 2 after the connection experiences earthquake damage. Table 1 shows that the CFRP installation improved the two-way shear strength of the connection by 38% (from $2.47\sqrt{f_c'}$ to $3.41\sqrt{f_c'}$). In addition, the capacity of SP-3 was about the same as that of SP-Control 3, implying that external CFRP sheets just as effective as the percentage of steel reinforcement.

The Effect of Seismic Damage on Punching Shear Strength

The influence of earthquake damage on the shear strength of interior slab-column connections can be evaluated by comparing the behavior of SP-Control 2 and SP-1. These were nominally identical specimens in every aspect. SP-Control 2 was tested to evaluate the shear strength of a connection that does not have any earthquake damage. SP-1 was tested after the connection accumulated damage during the lateral displacement excursions up to 1.25% drift. It is interesting to note that the earthquake damage in the connection region had no noticeable adverse effects on the two-way shear strength. It is believed that the low percentage of top flexural reinforcement in the connection region was the key parameter influencing the two-way shear strength of SP-Control 2 and SP-1. Wide flexural cracks were observed in SP-Control 2 and SP-1 at later stages of the loading (prior to failure). SP-1 had flexural cracks that formed during the lateral loading and these cracks opened wider during the punching shear tests. SP-Control 2, on the other hand, had no flexural cracks at the beginning of the punching shear test. However, with increasing loads, flexural cracks formed. At around 80-90% of capacity, the level of damage and crack widths around the critical perimeters of both specimens were comparable. As a result, both specimens failed at shear stress levels of about $2.5\sqrt{f_c'}$.

Use of CFRP stirrups

Most of the specimens tested in this study displayed limited deformation capacity. To improve the shear strength and deformation capacity of an earthquake-damaged slab-column connection, SP-2 was strengthened using CFRP stirrups. SP-2 maintained loads through large deformations (Fig. 7). A peak shear stress of $2.93\sqrt{f_c'}$ on the critical perimeter d/2 away from the column face was measured. On the critical section d/2 away from the CFRP–reinforced zone, the shear strength of concrete was equal to $1.83\sqrt{f_c'}$, which is equivalent to 53% of the capacity calculated through the use of Eq. 1.

Residual Capacity after Shear Failure

Since SP-Control 2, SP-Control 3, and SP-1 have exactly the same bottom (compression) reinforcement, they exhibited the same post-punching capacity. Immediately after punching shear failure, the primary resistance left in the slab is dowel action of the bottom reinforcement. The prying of the "dowels" can be seen in Fig. 8. The externally installed stirrups in SP-2 shifted the failure surface away from the column (Fig. 9), and therefore increased the number of bars contributing to the resistance following a punching shear failure. As a result, the post-punching capacity of SP-2 was high. In SP-3, after punching shear failure occurred, the well-anchored CFRP sheets acted as tension bands (Fig.10) and allowed the slab to carry substantial shear force through larger deformations. Reducing the capacity loss following a punching shear failure is highly desirable because it will reduce the load redistributed to other connections in an actual structure thereby reducing the risk of progressive collapse.

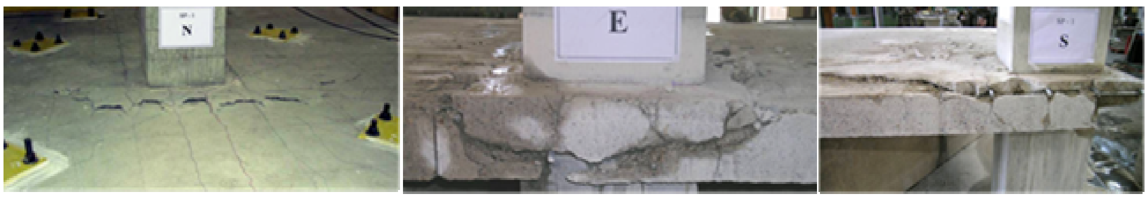


Figure 8. Failure surface of SP-1.

Figure 9. Failure surface of SP-2.

Figure 10. Failure surface of SP-3.

Conclusions

Experimental research on the two-third-scale models of an interior slab-column connection was conducted. The test results show that two-way shear strength was sensitive to the slab top reinforcement ratio within (c+3h) region. Punching shear strength of the undamaged control specimens with 0.5% and 1.0% top reinforcement ratio were only 63% and 85% of that estimated using ACI 318-05 expression ($V_c = 4 \times \sqrt{f_c^{'}} \times b_o \times d$).

Punching shear failure of all specimens was initiated by large flexural cracks that reduced the concrete contribution to shear strength and resulted in early punching shear failure. Damage due to 1.25% lateral drift did not affect the punching shear capacity of the specimen tested. Two methods for strengthening earthquake-damaged connections with 0.5% top reinforcement ratio were developed: (i) installation of the external CFRP stirrups and (ii) application of well-anchored CFRP sheets on the tension surface of the slab. In addition to increasing the punching shear strength of the slab-column connection, the rehabilitation methods developed in this study also improved the residual capacity after punching shear failure.

Acknowledgements

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