

NON-DUCTILE SLAB-COLUMN CONNECTIONS SUBJECTED TO CYCLIC LATERAL LOADING

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SUMMARY

Reinforced concrete flat slab floors are used throughout the world as an economical structural system for many building applications. In moderate and high seismic regions, a more rigid structural system, such as a shear wall or moment resisting beam column frame, is generally added to provide adequate lateral resistance. Nevertheless, the slab-column system must maintain its gravity load capacity even after numerous cyclic lateral displacements. Flat slab buildings with discontinuous bottom reinforcement are susceptible to progressive collapse if punching shear failure occurs at a connection. Many such failures have occurred in past earthquakes resulting in significant loss of life. Evaluation of older flat-slab buildings must include a realistic prediction of the response of the slab-column connections. Considerable research has been performed on connections with continuous slab reinforcement; however, there is a lack of data on the performance of slab-column connections with discontinuous reinforcement. The research presented in this paper involved the cyclic lateral loading of six slab-column connections with discontinuous slab reinforcing typical of flat-slab buildings built prior to 1970. Test parameters include the level of gravity load on the slab during cyclic testing, the slab flexural reinforcement ratio, and the use of bent-up bars. The test results are compared with those from prior studies. Based on these tests, punching shear failure does not appear to occur earlier than in equivalent specimens with continuous reinforcement, however, the consequences are significantly more severe. Conservatism in estimating the lateral drift at which punching failure occurs is therefore warranted.

INTRODUCTION

In the 1950s, the trend towards lighter and more flexible construction configurations led to increased usage of flat plate construction—particularly for medium to high rise office and residential buildings [1]. Reinforced concrete flat plate construction has been and continues to be used as an economical structural system for many buildings. In moderate and high seismic regions, flat plate structures are supplemented with either a moment frame or shear wall lateral resisting system. Today, ductile detailing for all structural connections, including for those which are "gravity load only", is a key concept learned initially as a result of the failures observed during the 1971 San Fernando earthquake [1]. All the "gravity load only" slab-column connections in a flat plate structure must maintain their capacity at the maximum

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displacement of the lateral system. During this lateral deformation the brittle failure mode of slab punching can occur. A punching shear failure, generated by the combination of gravity loading and seismically induced unbalanced moment, can occur with little or no warning and has resulted in the progressive collapse of these types of structures. In the 1985 Mexico City earthquake, 91 flat plate buildings collapsed and 44 were severely damaged due to punching failure [2].

Prior to the ACI code revisions in the 1970s, which began to reflect ductile detailing lessons learned, reinforcing for flat slab systems did not require continuity of top and bottom reinforcement. Top reinforcing, used for negative bending, could be completely curtailed away from the column supports. Bottom reinforcing was only required to extend into supports by 150 mm (6 inches). It is now well known that positive bending can occur at the face of supports during lateral displacements inducing a bond failure at these short embedment locations. Due to the inadequacies of the pre-1971 design codes, there is a need to understand the behavior of the structures designed to these codes to assist in determining their true seismic capacity/behavior.

This paper presents the results of tests performed on large scale slab-column connections, designed with pre-1971 non-continuous slab reinforcing. Six half-scale interior connections with varying reinforcing ratios, detailing and slab gravity loads were subjected to a cyclic lateral loading routine.

TESTING PROGRAM

Test Specimens

Each test specimen represents a half-scale model of an interior flat-plate slab-column connection (Figure 1). The specimens were designed to evaluate the influence of gravity load, slab reinforcement ratio, and bent-up bars in the connection region, on the seismic performance of interior slab-column connections.



Figure 1-Prototype Building and Test Specimen.

The slab-column connections were tested under combined gravity and cyclic lateral load. The gravity load applied to the slab during the test was equivalent to a loading on the full-scale structure of the total dead load plus 30 percent of the floor live load. This gravity load produced an effective shear stress on the

critical perimeter equal to 25 percent of the direct punching shear capacity of the concrete—defined by the ACI 318 Building Code [3]. Two specimens were subjected to increased gravity loading with the largest slab load representing a shear on the critical perimeter of 48 percent of the direct punching shear capacity.

The slab reinforcing details used in these half-scale specimens were typical of older flat slab construction in moderate and high seismic regions. In all specimens, the top slab reinforcement extended to one-third of the span and was not continuous through midspan. The bottom slab reinforcement was continuous at midspan, but discontinuous through the column, extending only 152 mm (3 inches) into the column support. The lack of continuous bottom reinforcement passing through the column may result in total collapse of the slab after punching failure. In order to prevent this condition in the laboratory tests, two continuous slab bottom bars were added transverse to the loading direction in all specimens except ND8BU. It was expected that these bars would not affect the specimen response significantly, except to prevent total collapse after punching. Details of the specimen reinforcement are provided in Figures 2 through 5. The six non-ductile interior slab-column connections tested as part of this program are:

- ND1C Control specimen
- ND4LL Identical to ND1C but with increased slab gravity load.
- ND5XL Identical to ND1C but with slab gravity load greater than ND4LL.
- ND6HR Identical to ND1C but with increased slab flexural reinforcement.
- ND7LR Identical to ND1C but with decreased slab flexural reinforcement.
- ND8BU Identical to ND1C but with additional bent-up bars at the connection.

The slab reinforcement layout for specimens ND4LL and ND5XL was identical to that of the control specimen, ND1C as shown in Figure 2. Specimen ND6HR had heavier slab reinforcement than the control specimen as shown in Figure 3. Because of the increase in slab top reinforcement, it was anticipated that this specimen would resist a greater lateral load than the control specimen. As a consequence, if all of the top reinforcement were curtailed at one third span as for the control specimen, it is likely that the negative moment induced at this section would exceed the cracking moment of the unreinforced slab, resulting in a premature flexural failure. In order to fully test the slab-column connection region, half of the top reinforcement was continued through midspan as shown in Figure 3. It should be noted that older flat slab buildings with heavy top reinforcement curtailment. Any retrofit scheme for such a structure should provide reinforcement to prevent this premature flexural failure.

Specimen ND7LR had lighter slab reinforcement than the control specimen as shown in Figure 4, while specimen ND8BU had a similar reinforcement layout except with additional bent-up bars added (Figure 5). These bent-up bars increased the top reinforcement ratio at the face of the column and the bottom reinforcement ratio at midspan. Again, half of the top reinforcement was continued to midspan to ensure that flexural cracking of the slab at the curtailment of top reinforcement would not cause premature flexural failure.

The specimens were designed using the ACI 318-63 Building Code, which did not require continuous bottom reinforcement through column lines. No shear reinforcing was included since the concrete shear capacity was adequate for the code defined ultimate conditions.



Figure 2-Slab Reinforcement for ND1C, ND4LL, and ND5XL.







Figure 4-Slab Reinforcement for ND7LR.



Figure 5-Slab Reinforcement for ND8BU.

Test Displacement Routine

The lateral displacement test routine used in this program was based on a lateral testing routine developed by PEER researchers based on previous research performed by Krawinkler [4]. The cyclic routine was

performed in two phases: Phase I consisted of both positive and negative cycling to 5 percent drift level (maximum capacity of the actuator); Phase II consisted of cycling up to 10 percent drift, but only in the positive drift direction (Figure 6). This protocol gradually increases the drift level from $\pm -0.1\%$ to $\pm -5\%$ in Phase I and from +7% to +10% in Phase II. To evaluate the loss of strength and stiffness after repeated loading of the structure, and to induce the type of damage expected in a seismic event, each drift level was repeated three times.



Figure 6-Lateral Displacement Routine.

Test Setup and Instrumentation

The specimens were tested as shown in Figure 7. A pin support, with two load cells to monitor column axial load and shear, was located at mid-height of the column below the slab. Three pin ended vertical load rods at each edge of the slab prevented vertical displacement of the slab, but allowed free lateral movement and rotation, thus simulating mid-span conditions in the direction of loading. The cyclic lateral displacement routine (Figure 6) was applied to a pin connection at the mid-height of the column above the slab via an actuator with an internal linear variable displacement transducer (LVDT) and an inline load cell.



Figure 7-Test Frame Configuration.

TEST RESULTS

Material Properties

Concrete

The concrete used to make the specimens was supplied by a local ready-mix company with a specified compressive strength of 24 MPa. Three 152×304 mm cylinders and two $152 \times 152 \times 533$ mm beams were fabricated and cured in the same conditions as each slab. These were tested at the same age as the slab-column connections. The resulting concrete properties are listed in Table 1. The concrete compressive strength in Table 1 represents the actual strength of the concrete at the time of testing, and not the design 28 day strength.

Specimen	ND1C	ND4LL	ND5XL	ND6HR	ND7LR	ND8BU
Compressive Strength, f _c (3 cylinders) (MPa)	29.6	32.3	24.1	26.3	18.8	39.2
Modulus of Elasticity, E _c (1 Cylinder) (GPa)	17.4	21.3	18.5	17.2	15.0	18.2
Modulus of Rupture, f _r (2 Beams) (MPa)	4.20	4.22	2.49	4.27	2.83	4.62
Poisson's Ratio, v (1 Cylinder)	0.23	0.24	0.29	0.20	0.23	0.22

Table 1-Concrete Material Properties

Steel Reinforcing

The reinforcing used in both the slab and column in each specimen was specified as Grade 60 Type 2 deformed bars. The slab reinforcing was 10 mm diameter deformed bars with nominal yield strength of 420 MPa.

Load-Drift Relationships

Relevant data collected during each specimen test are summarized in Table 2. These data include the initial gravity load supported by the column (row 1); the ratio between the initial gravity load and the ACI 318 shear capacity of the critical perimeter (row 2); the gravity load at failure (row 3)—different from row 2 due to redistribution to the load rods; the ratio between the failure gravity load and the shear capacity of the critical perimeter (row 4); the maximum horizontal load during hysteresis in the positive direction (row 5); the maximum horizontal load during hysteresis in the negative direction (row 6); the positive drift level at the maximum horizontal load (row 7); the negative drift level at the maximum horizontal load (row 8); the maximum drift level before failure (row 9); and the type of failure (row 10).

Specimen	ND1C	ND4LL	ND5XL	ND6HR	ND7LR	ND8BU
1) Initial Gravity Load, Vg (kN)	60.8	93.4	104.8	67.2	68.5	65.3
2) Initial Gravity/Shear Ratio, Vg/Vo	0.25	0.37	0.48	0.30	0.36	0.24
3) Gravity Load @ Failure, V _{gf} (kN)	55.1	70.9	101.2	65.5	49.0	71.3
4) Failure Gravity/Shear Ratio, V _{gf} /V _o	0.23	0.28	0.47	0.29	0.26	0.26
5) Maximum Positive Horizontal Load (kN)	28.6	31.2	22.7	40.6	18.8	41.5
6) Maximum Negative Horizontal Load (kN)	-30.9	-32.4	-23.7	-42.7	-21.8	-42.8
7) +Drift @ Maximum Horizontal Load (%)	3-5	3	1.5	3	3	3
8) – Drift @ Maximum Horizontal Load (%)	-3	-3	-1.5	-3	-3	-3
9) Max. Drift Attained Before Failure (%)	8	4	2	5	5	5
10) Type of Failure	Flexure/	Punch	Punch	Punch	Punch	Punch
	Punch					

Table 2-Specimen Test Data Summary

The load-drift relationships for each specimen are shown in Figures 8 through 13. These figures show the hysteretic response, the resulting backbone curve, and critical points identified on these curves.



Figure 8-Behavior Specimen ND1C



Figure 9-Behavior of Specimen ND4LL





Figure 11-Behavior of Specimen ND6HR



Figure 12-Behavior Specimen ND7LR

Figure 13-Behavior of Specimen ND8BU

Comparison of Backbone Curves

The backbone curves shown in Figures 8 through 13 represent the envelope of peak lateral loads supported by the specimen at each drift level. Figures 14 to 16 show comparisons of various backbone curves.

Gravity Load Effect

Figure 14 shows the comparison of the control specimen, ND1C, with the two nominally identical specimens subjected to increased gravity load, ND4LL and ND5XL. The initial response of all three specimens is similar, however, the maximum drift level decreases as the gravity shear ratio (Vg/Vo) increases. Specimen ND4LL achieves the same peak lateral load at a similar drift as the control specimen, but fails due to punching shear soon thereafter. Specimen ND5XL with the heaviest slab gravity load suffers punching shear failure before reaching the flexural capacity of the slab reinforcement.



Figure 14: Effect of Gravity Shear Ratio.

Because of the lack of continuous bottom reinforcement passing through the column, both punching shear failures would have lead to complete collapse of the slab and potential progressive collapse of floors below this level.

Reinforcement Ratio Effect

Figure 15 shows the comparison between backbone curves for three specimens with significantly different slab flexural reinforcement ratios. Specimen ND6HR has more slab reinforcing than the control specimen, ND1C, while specimen ND7LR has less (See reinforcement lavouts in Figures 2, 3 and 4). As expected, the peak lateral load capacity varies with the reinforcement ratio. The increased unbalanced moment in the stronger and stiffer ND6HR specimen results in punching shear failure during the first cycle to 5%lateral drift.

Effect of Bent-up Bars

Figure 16 shows the comparison between backbone curves for the control specimen, ND1C, the specimen with increased slab flexural reinforcement, ND6HR, and the specimen with bent-up bars, ND8BU. Because of their similar reinforcement ratios, specimens ND6HR and ND8BU resisted the same peak lateral loads at the same lateral drift level. Both specimens experienced punching shear failure during excursions to 5% lateral drift. The bent-up bars were effective at preventing progressive collapse of the slab since they are anchored into the bottom of the slab adjacent to the connection region.

Reinforcement Ratio Comparison



Figure 15: Effect of Flexural Reinforcement Ratio.



Figure 16: Effect of Bent-up Bar Reinforcement.

ANALYSIS

Current ACI Code Requirements

The ACI code establishes a method to determine the combination of direct shear and unbalanced moment on a slab-column connection that will cause a punching failure. According to the code, the shear stresses are evaluated at a critical section around the column. This critical section is located at a distance of d/2from the face of the column, where d is the average depth of the tensile (top) steel from the compression surface (bottom) of the slab. The code method assumes that the shear at the critical section is the combination of the direct shear (V_u) and a portion of the unbalanced moment (M_u) at the connection. The total maximum shear stress acting on the critical perimeter (υ_u) is determined from the following equation:

$$\upsilon_{u} = \frac{V_{u}}{A_{c}} \pm \frac{\gamma_{v} M_{u} c}{J_{c}}$$
 Eqn. (1)

where A_c is the area of the critical perimeter, J_c is the polar moment of inertia of the critical perimeter, γ_v is the proportion of the unbalanced moment assumed to be transferred as an eccentric shear stress, and c is the distance between the centroid and edge of the critical perimeter.

The concrete shear stress is limited to the smallest of three concrete stress equations given in ACI 318 Section 11.12.2.1. The slabs tested are controlled by the following equation:

$$v_{\rm c} = 1/3 \sqrt{f_c}$$
 Eqn. (2)

where the concrete strength f'_c is in MPa.

Comparison with ACI

The ACI code approach was applied to each of the slab-column specimens. Values and results for the peak shear, unbalanced moment, and shear stresses around the critical perimeter for the retrofitted specimens are presented in Table 3.

Spec.	Unbalanced	Direct	Ultimate	Computed	Shear	Flexural	Flexural	Moment
	Moment	Shear	Shear	Concrete	Ratio	portion of	Moment	Ratio
		Force	Stress	Stress		Applied	Capacity	
						Moment		
(+/-)	M_u	\mathbf{V}_{u}	v_{u}	$v_n = v_c$		$\gamma_{\rm f}M_{\rm u}$	M_{f}	
					ν_u/ν_n			$\gamma_fM_u\!/M_f$
	(kN-m)	(kN)	(MPa)	(MPa)		(kN-m)	(kN-m)	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
ND1C+	43.6	59.1	1.55	1.81	0.86	26.2	15.7	1.66
ND1C-	-47.0	52.0	1.59	1.81	0.88	-28.2	15.7	1.79
ND4LL+	47.6	73.9	1.64	1.89	0.87	28.6	15.7	1.82
ND4LL-	-49.3	76.3	1.82	1.89	0.97	-29.6	15.7	1.89
ND5XL+	34.6	98.6	1.51	1.63	0.92	20.7	15.5	1.34
ND5XL-	-36.1	95.6	1.63	1.63	1.00	-21.6	15.5	1.39
ND6HR+	61.8	65.4	2.06	0.85	2.42	37.1	23.0	1.61
ND6HR-	-65.0	65.9	2.14	0.85	2.51	-39.0	23.0	1.70
ND7LR+	28.6	57.6	1.16	0.72	1.60	17.2	8.2	2.09
ND7LR-	-33.3	52.0	1.23	0.72	1.71	-20.0	8.2	2.43
ND8BU+	63.3	69.4	2.13	1.04	2.04	38.0	23.3	1.63
ND8BU-	-65.2	65.8	2.15	1.04	2.06	-39.1	23.3	1.68

 Table 3 - ACI 318 Comparison: Laterally Tested Specimen

The first column in Table 3 contains the slab-column specimen designation. Each specimen has two rows; the first row lists results for the peak lateral load in the positive direction, while the second row refers to the peak lateral load in the negative direction. The second column is the total unbalanced moment transferred between the column and the slab at the peak lateral load. It is calculated by multiplying the peak lateral load by the story height of the specimen (1524 mm). Column three lists the

total gravity load being carried by the column at the same time as the peak lateral load. This value is approximately the same as the total direct shear force acting on the slab at the critical perimeter.

The next three columns list the shear stress capacities of the slab at the critical perimeter. In column four, the ultimate shear stress is the maximum shear stress acting on the critical perimeter due to the loading condition present during the application of the peak lateral load. This stress is based on the linear shear stress distribution due to the gravity load and the portion of the unbalanced moment carried by an eccentric shear, $\gamma_v M_u$. The concrete stress, v_c , (column 5) is calculated using the controlling formula in the ACI Building Code (Eqn. (2)) for the nominal shear stress. Since these slabs do not contain shear reinforcement, this represents the nominal shear stress capacity of the slab-column connection at the critical perimeter, v_n . Column 6 is the ratio of the maximum shear stress induced by the loading condition at the peak lateral load to the nominal shear capacity of the connection. A value of 1.0 for the shear ratio in column 6 would indicate that the connection is on the verge of punching shear failure according to the ACI Building Code.

Slab-column connections may also fail due to flexural failure. According to the ACI Code, the portion of the unbalanced moment not carried by shear is carried by flexure, $\gamma_f M_u$. These values are listed in column 7 of Table 4. If this moment exceeds the flexural moment capacity of a slab width of $c_2 + 3h$ centered on the column, M_f , then the ACI Code predicts a flexural failure. Values of M_f are listed in column 8. Lastly, the moment ratio (column 9) is the ratio of the unbalanced moment resisted by flexure to the nominal moment capacity of the slab within $c_2 + 3h$. A value in column 9 greater than unity indicates that the connection is carrying a load that is greater than that predicted by the ACI Building Code and flexural failure should result.

Specimens ND1C, ND4LL and ND5XL Comparison

Specimen ND1C was observed to fail first in flexure at 6% drift followed by punching failure at 9% drift. The average moment ratio of 1.73 and an average shear ratio of 0.87 are consistent with the observed behavior. The extensive crack formation during flexural concrete deterioration lead to the shear failure at an induced shear stress less than that predicted by Eqn 1. The average moment ratio of 1.86 and average shear ratio of 0.92 for specimen ND4LL would also indicate this type of behavior. The peak lateral load occurred at a drift of 3% followed by a reduced lateral load at a drift of 4% just prior to punching. For specimen ND5XL, the average shear ratio of 0.96 indicates that punching shear failure was imminent. Indeed, punching shear failure occurred prior to reaching the flexural capacity of this connection.

Specimens ND6HR and ND7LR Comparison

The failures of specimens ND6HR and ND7LR show that an increase in the flexural stiffness of a slabcolumn connection can induce a higher shear demand. The average shear ratio of the stiffer ND6HR was 2.47 while the average shear ratio of ND7LR was 1.66. Specimen HD7LR maintained one complete cycle of 5% drift before punching, however, ND6HR punched at negative 4% drift in the first cycle of the 5% drift regime.

Specimens ND6HR and ND8BU Comparison

At the column face, these two connections have similar reinforcing ratios. The effect of this reinforcing results in similar demand for both shear and bending. An average shear ratio of 2.05 and an average moment ratio of 1.66 for specimen ND8BU are both similar to those for specimen ND6HR: 2.47 and 1.66 respectively. The hysteretic responses for these specimens are virtually identical, however, the bent-up bars in specimen ND8BU provided a mechanism to prevent catastrophic collapse once punching shear failure occured.

CONCLUSIONS

Based on the results of this cyclic lateral loading test program on interior slab-column connections with discontinuous slab reinforcement detailing, the following conclusions were drawn:

- 1. Slab-column connections with discontinuous slab reinforcement perform similar to those with continuous reinforcement until punching shear failure. After punching failure, connections without continuous bottom reinforcement passing through the column will suffer complete collapse, which may lead to progressive collapse of the floors below the initial failure. The exception to this observation is that bent-up bars passing through the column as top reinforcement, but anchored as bottom reinforcement in the slab, were able to prevent collapse after punching failure.
- 2. Increased gravity load on the slab during cyclic lateral loading results in a significant reduction in lateral drift capacity before punching failure. For heavy slab loading conditions, punching failure can occur before reaching the lateral load capacity of the control connection.
- 3. Reduction of gravity load on the slab through alteration of the building occupancy and function could serve as an economical retrofit technique.
- 4. Connections with increased slab flexural reinforcement will support greater lateral loads, but the increased eccentric shear transfer may result in premature punching shear failure.

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