Effect of slab flexural reinforcement and depth on punching strength

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Abstract

Recommendations are presented for proposed revisions to the punching shear provisions of ACI 318-14 in order to recognize the limitations imposed on punching shear capacity by low amounts of slab flexural reinforcement and by slab depth effects. The influence of those two factors is investigated by examining relevant experimental results from tests on slab-column assemblages and slab systems.

Keywords

Concrete design codes, depth effect, flat slabs, moment transfer, punching shear, reinforcement ratio, size effect, slab-column connections

1 Introduction

The nominal punching shear strength for normal weight concrete, $v_c = 4\sqrt{f'_c}$, in psi ($0.33\sqrt{f'_c}$ in MPa) specified in Eq.(a) in Table 22.6.5.2 of ACI 318-14 for two-way members without shear reinforcement, has remained essentially unchanged since its introduction in the 1963 edition of ACI 318. This value, and the critical section associated with it, were developed by ACI-ASCE Committee 326 (1962), largely based on the work of Moe (1961) along with results from earlier investigators. Most of the analyzed results were for slab to interior column connection assemblages where, for the slab, the effective depth was limited to 4.5 in. (114 mm) or less, the negative moment flexural reinforcement ratio was 1.06 % or greater, and concrete strengths ranged from 3,000 to 4,000 psi (20.7 to 27.6 MPa). Committee 326 noted that it is not economically feasible to test slab systems to determine punching shear strengths, and that because the punching problem can be considered a localized condition involving only a portion of the slab system, it is reasonable to base the nominal punching shear strength on the results of slab-interior column assemblage tests. In the expression Moe derived, v_c is a function of ϕ_o, which equals the shear force at punching, V_{test}, divided by the shear force for flexural failure, V_{flex}. While it is easy to calculate V_{flex} for the slab of a slab-interior column assemblage based on yield-line theory, it can be difficult to calculate V_{flex} for practical slab systems. Further, yield line theory is generally not used for design for flexure in American practice. While Committee 326 saw ϕ_o as an important variable for analyzing slab-column data, they reasoned it was not an important variable for slab systems because the shear capacity of the system should exceed its flexural capacity. They derived the $4\sqrt{f'_c}$ value by taking ϕ_o as unity and assuming that v_c should approach infinity as the ratio of the column side dimension to the effective slab depth decreased.

Since the 1960s American concrete design practice has evolved considerably with reinforcing steel grades of 100 (690 MPa) or more being used in slabs and concrete strengths often being greater than 4,000 psi (27.6 MPa). For typical office and residential building designs, column connections are routinely stressed to ϕv_c and slab column strip negative moment flexural reinforcement ratios are 1.0% or less. Further, that reinforcement is commonly placed at a uniform spacing in the slab region around the column, even though the slab moment is a maximum at the column. This uniform distribution of steel is considered permissible due to the ability of slabs to redistribute moments over a significant width. Moe (1961) conducted two series of tests where the flexural reinforcement was concentrated in the vicinity of the interior column while the reinforcement ratio for the slab as a whole was kept constant. Moe found that such concentration did not increase the punching shear strength but did increase the flexural rigidity of the slab.

For slab to interior column connections transferring shear only, the results of tests (Criswell, 1974; Muttoni, 2008; Peiris and Ghali, 2012; Tian et al., 2008, and Widianto et al., 2009) and the compilation of a comprehensive database of existing test results (Ospina et al., 2011, 2012) have shown that the punching shear strength provisions of ACI 318-14 can be non-conservative in two situations: (1) where the slab flexural tension Grade 60 (414 MPa) reinforcement in the immediate vicinity of the column is less than 1%; and (2) where slabs have effective depths greater than 10 in. (250 mm). The objective of this paper is to summarize the test data that justify ACI 318-14 change proposals that address those two deficiencies and report the basis of such proposed code changes.

2 Slabs with flexural tension reinforcement ratios less than 1%

2.1 General principles

For one-way action, it is easy to distinguish between a flexural and a shear failure. Shear failures seldom develop once the rotations associated with flexural yielding occur. The behavior for two-way action is different. It is difficult to distinguish visually between a "pure" punching failure and a "flexure-driven" punching failure. Under gravity loading, inclined cracking develops around the perimeter of the loaded area or column, and within the body of the slab, at a shear stress of about $2\sqrt{f'_c}$ psi ($0.165\sqrt{f'_c}$ MPa) even though a "pure" punching failure does not occur until a stress of about $4\sqrt{f'_c}$ psi ($0.33\sqrt{f'_c}$ MPa) or more. The increase in strength between inclined cracking and failure is resisted, in large measure, by aggregate interlock along the inclined crack. This situation is illustrated in Fig.1 which shows the inclined crack and relationship between crack width and slab rotation ψ.

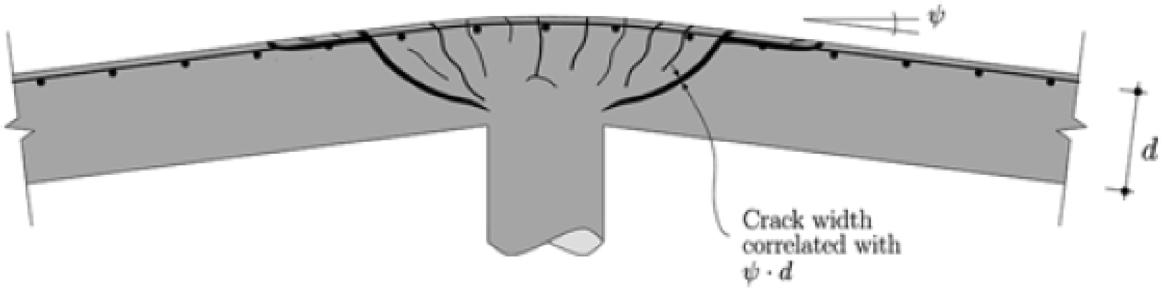

Figure 1: Correlation between inclined crack opening, slab thickness, and rotation (Muttoni, 2008).

As the flexural tension reinforcement in the slab around the column decreases, the flexibility of the connection at a given shear stress level increases. Experience shows that where Grade 60 (414 MPa) reinforcement is less than about 1%, yielding of this reinforcement adjacent to the column increases rotations. Consequent crack openings permit sliding along the internal inclined crack. "Flexure-driven" punching failures may occur at shear stress levels less than $4\sqrt{f'_c}$ psi ($0.33\sqrt{f'_c}$ MPa) (Widianto et al., 2009; Tian et al., 2008; Muttoni, 2008; Dam and Wight, 2016). Slab rotations that develop before such failures are often only marginally greater than those associated with a "pure" punching failure.

The nominal shear strength v_c must be applicable for moment and shear transfer as well as for shear transfer only. Shown in Fig. 2 are the stresses inferred from measured steel strains across the slab width with increasing load for a slab-interior column assembly test (Hawkins et al., 1989) with 0.96% tension reinforcement in the slab, with both shear and moment transferred to the column, and with shear reinforcement surrounding the bars passing through the column.

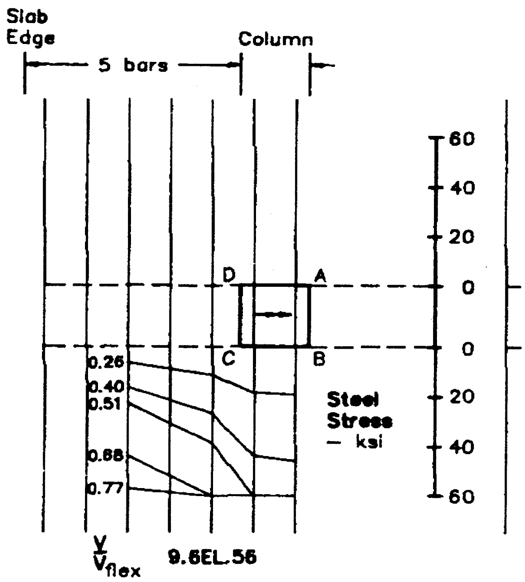

Figure 2: Distribution of steel stresses across slab width (Hawkins et al., 1989). (1 ksi = 6.895MPa)

In Fig. 2, V is the shear transferred to the column and V_{flex} is the shear for yielding across the full width of the slab. The slab reinforcement passing through the column began yielding at about 50% of V_{flex}. A punching failure occurred at about 80% of V_{flex} after yielding had spread to the first of the flexural bars passing outside the column. For similar connections with 0.6% slab reinforcement, without shear reinforcement, the failure mode was similar but the shear stress at failure was as low as $3.2\sqrt{f'_c}$ psi ($0.26\sqrt{f'_c}$ MPa).

For a slab-interior column assembly, yielding of the slab reinforcement in the vicinity of the column must occur before V_{flex} can be developed and local yielding can result in a "flexure-driven" punching failure. In a real-life slab, where continuity effects would make the calculation of V_{flex} a rather difficult task, it is sensible to deal instead with the shear force associated with the flexural strength of the slab locally around the column, V_{ly}. The issue then becomes what is the appropriate way to determine V_{ly} of the slab for comparisons with the measured slab punching shear strength.

The critical shear crack theory (CSCT) (Muttoni, 2008) analyzes conditions for sliding along the internal shear crack. Its use, in accordance with procedures of the Swiss Concrete Code, gives good agreement with available data for slab-column connection tests exhibiting both "pure" and "flexure-driven" punching failures and having a wide range of properties, including low reinforcement ratios and effective depths greater than 10 in. (250 mm). However, use of the CSCT is relatively complicated and its application to slab systems with a wide variety of column layouts and span lengths can be difficult. While such complications can be readily addressed through the use of computer analyses, the shear strength of the slab-interior column connection often controls slab depth and system span length choices. Therefore, a procedure that retains the simplicity of the $4\sqrt{f'_c}$ psi ($0.33\sqrt{f'_c}$ MPa) approach of ACI 318-14, while ensuring that "flexure-driven" punching failures are avoided, is preferable.

For slab-interior column connections, calculated strengths are similar to those of the CSCT for low reinforcement ratios, provided the punching shear capacity at the column is limited to the shear associated with the local yield strength of the tension reinforcement in the slab surrounding the column. For most laboratory test specimens, the shear associated with that local yield strength, V_{ly}, is a function of the distance from the column face to the edge of the test slab. However, in practice, at interior column connections, there is often no defined slab edge and V_{ly} can be approximated as slightly greater than 8 m (Peiris and Ghali, 2012) where m is the nominal flexural strength of the slab per unit width for the reinforcement within $1.5h$ of the column perimeter. The corresponding local yield strength for edge and corner columns can then be approximated by reducing the constant 8 in direct proportion to the ratio of the contact perimeter between the slab and the column for the non-interior column versus the same contact length for the same column as an interior column.

The local flexural strength V_{ly} can be approximated as:

$$V_{ly} = 0.2\,\alpha_s\,m \qquad (1)$$

where α_s, as defined in 22.6.5.3 of ACI 318-14, is 40 for interior columns, 30 for edge columns, and 20 for corner columns. Those values for edge and corner columns are for slabs extending the full length of the column side dimensions. Note Equation 1 has units of force.

2.2 Slab-interior column connections without shear reinforcement and transferring shear only

The application of the foregoing concepts to slab-interior column connections transferring shear only are discussed in greater detail in Ospina and Hawkins (2013). Shown in Fig. 3 is the agreement between measured and computed strengths for available slab to interior column connection data when V_{test} is limited to V_{flex} and where the connection is transferring shear only. V_{flex} in Fig. 3 is the theoretical yield line capacity for the test specimen. On the ordinates, k_v is a correction for slab depth effect as discussed later in the paper.

Shown in Table 1 are properties for all the specimens, displaying either flexure-driven or pure punching failures, where the abscissa values of Fig. 3 are less than 5.0. Test specimen properties were taken directly from the NEES databank (Ospina et al., 2011 and 2012). The 45-degree line in Fig. 3 captures reasonably well the expected failure load of test slabs displaying flexure-driven punching failures, confirming that punching for these slabs can in fact occur at a stress below $4\sqrt{f'_c}$ psi ($0.33\sqrt{f'_c}$ MPa), as noted, among others, by Widianto et al. (2009) and Peiris and Ghali (2012). For test slabs displaying pure punching failures, Fig. 3 shows that the current ACI design provisions provide a reasonable lower bound for the selected test data. It is worth noting that not until the ACI 445 punching test databank was developed (Ospina et al., 2011), test slabs displaying flexure-driven punching failures were commonly disregarded in earlier punching test databank compilation and evaluation efforts on the assumption that these slabs do not provide an explanation for pure punching failure. Ignoring this important experimental evidence hides the fact that slabs with low amounts of flexural reinforcement can in fact punch at loads below the basic value defined in traditional design codes.

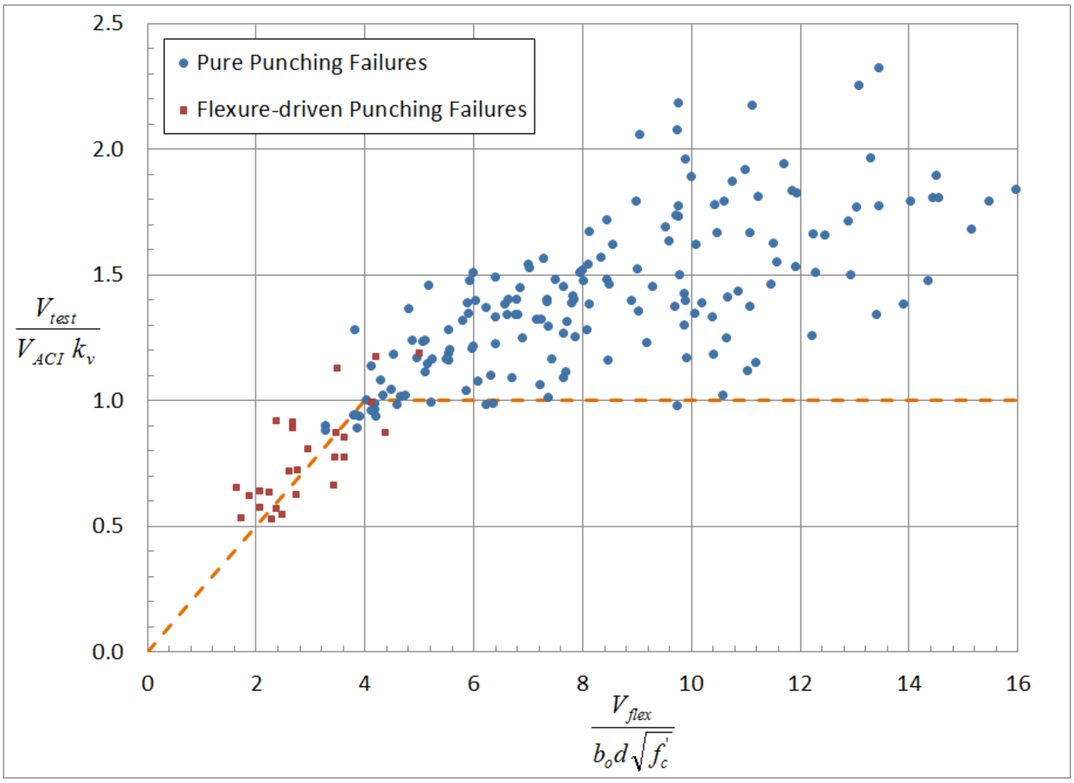

Figure 3: Correlation of measured and calculated strengths for slab-interior column assemblages transferring shear only.

Table 1: Properties of slab-interior column subassemblage tests transferring shear only, with $V_{flex}/(b_o d\sqrt{f'_c})<5$

Researcher	Test ID	d_{ave} (mm)	c_1* (mm)	ρ_{ave}	a (mm)	f_{c1} (MPa)	fy (MPa)	Failure Mode
Widianto et al	G0.5	126	406.0	0.0049	661	29.8	421	F
Widianto et al	G1.0	126	406.0	0.0105	661	26.7	421	F
Guandalini	PG-2b	210	260.0	0.0025	1250	40.5	552	F
Guandalini	PG-3	456	520.0	0.0033	2320	32.4	520	P
Guandalini	PG-4	210	260.0	0.0025	1250	32.2	541	F
Guandalini	PG-5	210	260.0	0.0033	1250	29.3	555	F
Guandalini	PG-8	117	130.0	0.0028	625	34.7	525	F
Guandalini	PG-9	117	130.0	0.0022	625	29.3	525	F
Guandalini	PG-10	210	260.0	0.0033	1250	28.5	577	F
Oliveira et al	L1c	107	120.0	0.0098	990	54.3	749	P
Ospina et al	SR-1	120	250.0	0.0084	710	35.0	430	P
Matthys & Taerwe	R3	86	117.8	0.0179	375	33.3	589	F
Ghannoum	S1-U	100	225.0	0.0096	956	35.3	445	P
Ghannoum	S2-U	100	225.0	0.0096	956	54.2	445	P
Ghannoum	S3-U	100	225.0	0.0096	956	63.7	445	P
Hallgren	HSC9	202	196.3	0.0036	1075	79.9	634	F
Ramdane	1	98	117.8	0.0058	611	82.7	550	P
Ramdane	6	98	117.8	0.0058	611	95.3	550	P
Urban	P 1/1-0.8	95	200.0	0.0083	700	33.3	414	P
Urban	Pd 1/1-0.8	104	320.0	0.0076	640	35.0	414	F
Marzouk & Hussein	HS1	95	150.0	0.0049	675	63.7	490	F
Marzouk & Hussein	HS2	95	150.0	0.0084	675	66.5	490	P
Marzouk & Hussein	HS5	125	150.0	0.0064	675	64.6	490	P
Marzouk & Hussein	HS11	70	150.0	0.0096	675	66.5	490	F
Marzouk & Hussein	HS15	95	300.0	0.0148	600	67.5	490	P
Rankin & Long	1C	54	100.0	0.0042	270	26.1	530	F
Regan	I/6	79	200.0	0.0075	815	20.6	480	P
Regan	I/7	79	200.0	0.0080	815	28.5	480	P
Swamy & Ali	S-1	100	150.0	0.0056	770	35.0	462	P
Criswell	S2075-1	121	254.0	0.0079	889	30.8	331	F
Criswell	S2075-2	122	254.0	0.0078	889	27.6	331	F
Criswell	S4075-1	127	508.0	0.0075	889	25.3	331	F
Criswell	S4075-2	124	508.0	0.0077	889	30.6	331	F
Criswell	S4150-1	125	508.0	0.0152	889	33.7	331	P
Criswell	S4150-2	125	508.0	0.0152	889	33.9	336	P
Ladner	M	109	177.5	0.0120	487	39.6	541	P
Schaeidt et al	P1	240	392.7	0.0131	1075	26.2	544	P
Manterola	P2-S1	107	250.0	0.0106	1375	32.2	304	P
Manterola	P3-S1	107	450.0	0.0106	1275	28.2	304	F
Manterola	P2-S2	107	250.0	0.0106	1375	31.4	324	F
Manterola	P3-S2	107	450.0	0.0106	1275	30.3	324	F
Manterola	P1-S3	107	100.0	0.0073	1450	37.7	324	P
Manterola	P3-S3	107	100.0	0.0037	1450	37.3	324	F
Manterola	P2-S4	107	250.0	0.0047	1375	29.7	451	F
Manterola	P3-S4	107	450.0	0.0047	1275	32.5	451	F
Moe	M1A	114	305.0	0.0152	738	19.8	481	F
Kinnunen & Nylander	IA30d-32	123	235.6	0.0054	705	24.2	448	P
Kinnunen & Nylander	IA30d-33	125	235.6	0.0053	705	24.5	462	P
Elstner & Hognestad	A-1d	117	254.0	0.0118	762	35.0	332	P
Elstner & Hognestad	A-2b	114	254.0	0.0255	762	18.5	321	F
Elstner & Hognestad	A-4	117	355.6	0.0118	711	24.8	332	P
Elstner & Hognestad	A-8	114	355.6	0.0255	711	20.8	321	F
Elstner & Hognestad	A-13	121	355.6	0.0055	711	24.9	294	F
Elstner & Hognestad	B-2	114	254.0	0.0050	762	45.2	321	F
Elstner & Hognestad	B-4	114	254.0	0.0101	762	45.3	303	P

Notes: See NEES databank for specimen type. c^*_1 = column width (for specimens with a circular column, the column "width" is that of an equivalent square column with the same perimeter of the circular column). a = shear span. f_{c1} = uniaxial compressive strength of concrete prism (see NEES databank for conversion factors linking f_{c1} to concrete cylinder and cube strengths reported by researchers). f_y = yield strength of slab flexural reinforcement. Failure mode: P = pure punching; F = flexure-driven punching. (1,000 psi = 1 ksi = 6.895 MPa; 1 inch = 25.4 mm; 1 ft = 0.305 m)

2.3 Slab-interior column connections with shear reinforcement and transferring shear only

Recognition that there can be "flexure-driven" punching shear failures also implies that the nominal shear strength of slabs with shear reinforcement may be limited by the nominal shear strength associated with the local yielding of the slab flexural tension reinforcement at slab-column connections and in areas of concentrated load. From the results reported in Stein et al. (2007) and Dam and Wight (2016) it is apparent that the V_{ly} limit in Eq. 1 is also applicable to slabs with shear reinforcement where connections are transferring shear only. However, it has also been shown that while provision of shear reinforcement in excess of that required to achieve nominal flexural strength does not increase the shear capacity of a connection, such shear reinforcement does increase the rotations achieved at the slab-column connection before the flexure-driven punching failure occurs (Stein et al., 2007).

2.4 Prestressed concrete slabs

In laboratory tests on prestressed slab-column connections no flexure-driven punching failures have been reported. Theoretically, when the foregoing model for flexure-driven punching failures is extended to prestressed slabs, such failures are possible for slabs with low prestress levels and distributed tendons in both orthogonal directions. However, for the detailing requirements of ACI 318-14, (a minimum level of prestress of 125 psi (0.86 MPa), two tendons passing through the column in orthogonal directions, and minimum bonded reinforcement in orthogonal directions), coupled with the customary use of banded tendons in one direction and distributed tendons in the other, the flexural capacity at the slab-interior column connection consistently exceeds the shear capacity of the same connection. Hence, there is currently no need to bring up the limitation proposed here for prestressed concrete slabs.

3 Slab to column connections transferring moments and shears

The two-way strength provisions of 22.6.5.2 of ACI 318-14 are also used to calculate shear strengths for slab to column connections transferring moments as well as gravity load shears. Therefore, any changes to 22.6.5.2 must be applicable when there is moment as well as shear transfer. For edge and corner columns transferring moment perpendicular to the edge, the maximum modified values of γ_f in Table 8.4.2.3.4 of ACI 318-14 permit the designer in many cases to ignore the effects of moment transfer-induced shear stresses, provided v_{ug} values are limited. Because the v_{ug} limits are greater than most v_{ug} values likely in design, the effects of changes to 22.6.5.2 on moment transfer strengths need to be considered primarily for interior column connections and edge column connections transferring moments parallel to the edge only.

ACI Subcommittee 445C is developing databases of existing results for interior column to slab connections transferring moments and edge column to slab connections transferring moments parallel to the edge. For interior columns results for tests on over 160 isolated slab-interior column assemblies without shear reinforcement and 23 slab-column frames with at least one interior column have been identified. For similar V_{ug}/V_c and ρ values, the ratios of measured to ACI 318-14 calculated strength decreased as the slab depth increased. However, there are no data for slabs with effective depths greater than 5.0 in. (127 mm). As can be seen from the results shown in the tables in Appendix A, the ratios of measured to ACI 318-14 calculated strength decreased as the V_{ug}/V_c ratio increased and as the tension reinforcement ratio for the slab decreased. For 13 results, from seven different investigations, all of which had $\rho_{column\ strip} \leq 0.8\%$, the ratios of measured to ACI 318-14 calculated strength were less than unity.

The limitation of v_u, calculated using the expression in R8.4.4.2.3 of ACI 318-14, to the lesser of v_c from Table 22.6.5.2 or $V_{ly} / b_o d$ from Eq.1, was examined using the results for the specimens in the database where the reinforcement ratio within lines *1.5h* on either side of the column was 1.1% or less. The results of that examination are included in Appendix A and illustrated graphically in Fig. 4. There were 48 interior column to slab assemblies in that selected database and 12 frames with at least one interior column. The assembly tests were from 15 different studies and the frame tests were from four different studies. For the existing ACI 318-14 provisions there were seven assembly tests and two frame tests where v_u was less than v_c with the lowest result being 0.79. With the revised limit on v_c per Eq. 1 for connections transferring shear only, there were only two results less than 1.00 and the lowest result was 0.95. Clearly, the implementation of an additional limit on v_c per Eq.1 significantly improved the agreement between calculated and measured strengths. It also improved the agreement between the observed and predicted mode of failure.

Shown in Fig. 4 are ratios of test to calculated strength on the vertical axis versus, on the horizontal axis, ratios of the moment transferred to the column divided by the shear transferred simultaneously times the side length of the column. Because all columns for the specimens plotted were square, the notation for the side length of the column is c rather than c_1 or c_2 as used in ACI 318-14. Figure 4(a) is for the existing ACI 318-14 provisions and Fig. 4(b) is for the proposed code revisions. Where there is moment transfer, the predicted strength can be controlled by either the shear strength of the connection (shear controlled) or the adequacy of the reinforcement within lines *1.5h* on either side of the column (moment controlled or flexure-driven controlled). In both Fig. 4(a) and 4(b), the former case is shown by solid symbols and the latter case by open symbols. The test-to-calculated values for both Figs. 4(a) and 4(b) are taken from Tables A1.2 or A2.2 in Appendix A. For Fig. 4(a) the test-to-calculated value is the greater of $(V_T/ V_o + M_T/ M_o)$ and $(\gamma_f M_T/ M_R)$. For Fig. 4(b) the test-to-calculated value is the greater of $(V_T/ V_o + M_T/ M_o)$, $(V_T / V_{ly} + M_T/ M_o)$, and $(\gamma_f M_T/ M_R)$. The proposed code provision affects the shear strength limitation only and from a comparison of Figs. 4(a) and 4(b) it is clear that the proposed revision improves the agreement between test and calculated values, especially for low *M/Vc* ratios.

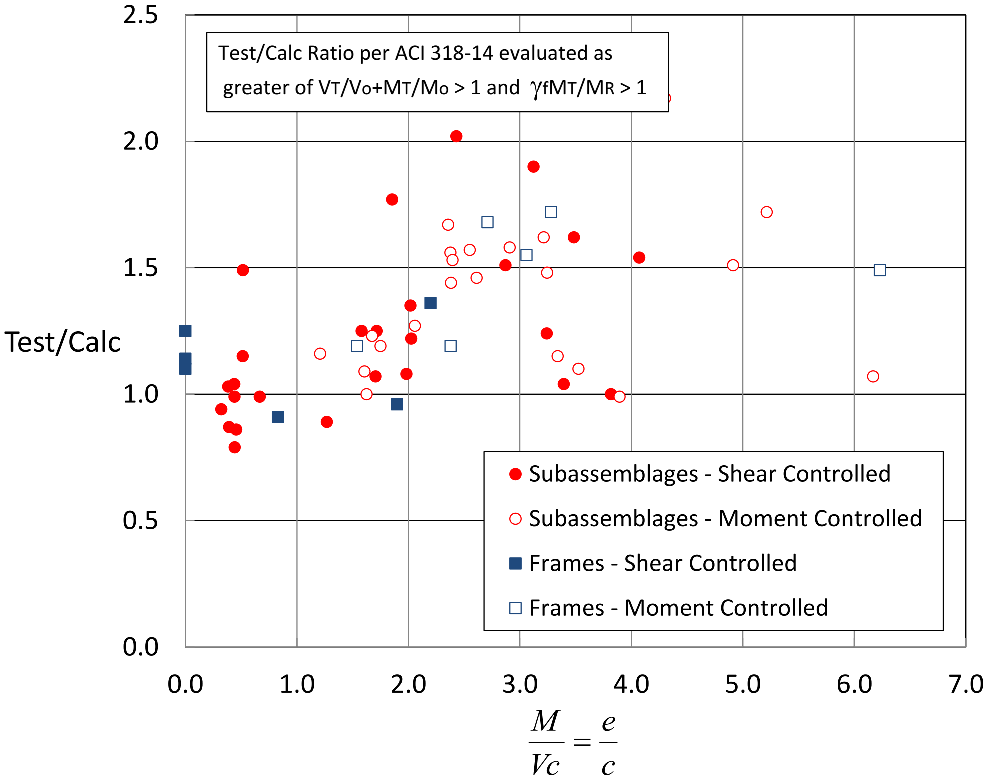

Figure 4(a): Moment transfer strengths per ACI 318-14 for data in Appendix A.

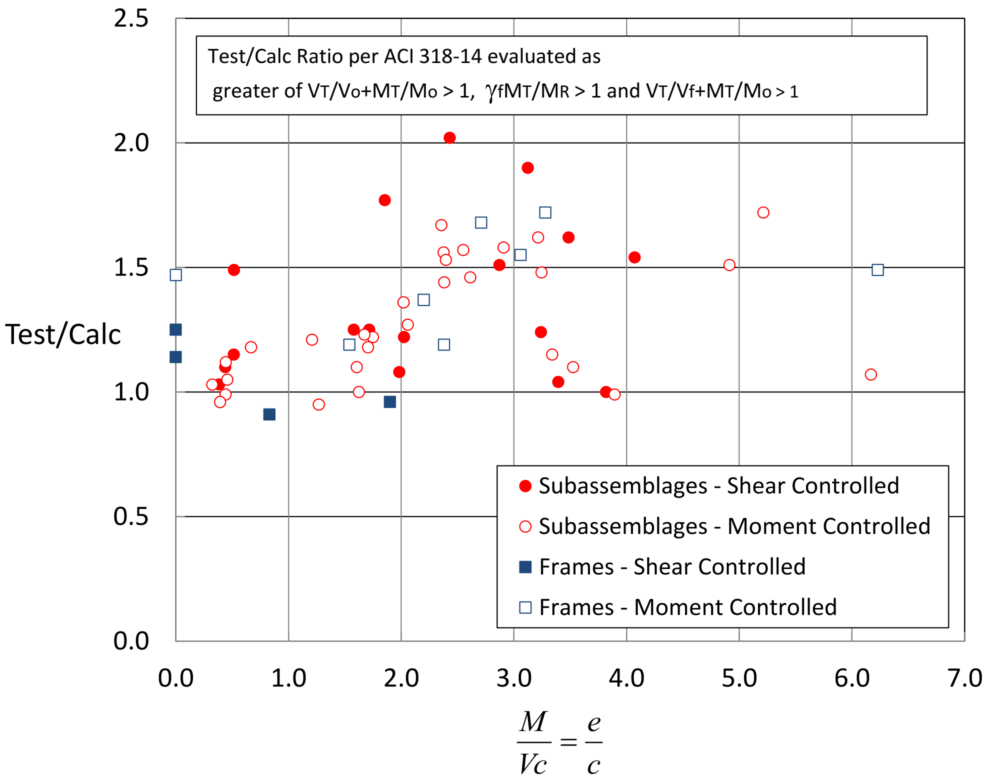

Figure 4(b): Moment transfer strengths per proposed revision for data in Appendix A.

4 Code limitation for low slab flexural reinforcement ratios

4.1 Lessons from design examples

As documented in Appendix A, the impact on design of the limitation of v_u to the lesser of $\phi\ v_c$ or $\phi\ v_{ly}$ for slab-column connections without shear reinforcement and the lesser of $\phi(v_c/2+ v_s)$ or $\phi\ v_{ly}$ for slab-column connections with shear reinforcement was examined for two typical structures designed by a leading structural engineering firm. One structure had 28 in. (711 mm) square columns, orthogonal spans of 30 ft (9.14 m) and a slab with an overall depth of 10in. (250 mm). The second structure had 12 in. (305 mm) square columns, orthogonal spans of 24 ft (7.32 m) and a slab with an overall depth of 8 in. (203 mm). For both structures, the design moments and shears to be transferred to the columns, the number of bars and their sizes for the column strip and the middle strip for a typical interior span, an end span with an exterior column transferring moment normal to the edge and a corner column (end span in orthogonal directions), were provided.

In both designs, the column strip reinforcement at the interior column was less than 1%. For the edge and corner columns, no changes were needed to the prior design with the proposed limitation on v_u. For both designs, revisions to the detailing of the reinforcement in the vicinity of the interior column were needed. For the 30 ft (9.14 m) span structure, there was a need to change the uniformly spaced slab tension reinforcement so that there was slightly more reinforcement within lines *1.5h* on either side of the column. With that adjustment, however, the spacing of the remaining tension reinforcement in the column strip outside the column area was still reasonable. For the 24 ft (7.32 m) span structure, shear reinforcement was required to meet existing ACI 318-14 provisions. To satisfy the proposed revision to the v_u limit, again slightly reduced reinforcement spacing within lines *1.5h* on either side of the column was needed. The results of these design analyses suggest that, for new construction, rather than requiring a calculation of V_{ly}, it is more appropriate to specify a minimum amount of reinforcement within lines *1.5h* on either side of the column or concentrated load area. For evaluations of the strength of existing construction, a calculation of V_{ly} may still be needed.

4.2 Reinforcement requirements for slabs without shear reinforcement

For slabs without shear reinforcement the ρf_y value required to meet the nominal punching shear strength specified in Table 22.6.5.2 of ACI 318-14 can be derived as follows:

For interior column connections, $V_{ly} = 8\ m = 8\ \rho f_y d^2$ (2)

In Eq. 2, m has been taken as $\rho f_y d^2$, which is a reasonable simplification for slabs.

For a punching failure, $V_c = 4\lambda\sqrt{f'_c}(b_o d)$ in psi ($0.33\lambda\sqrt{f'_c}(b_o d)$ in MPa) (3)

For Eq.1 to not control, $\rho f_y \geq \lambda\sqrt{f'_c}(b_o/2\ d)$ (4)

so that $\rho f_y \geq 2\lambda\sqrt{f'_c}(c_1/d+1)$ for square columns with side length c_1 (5)

and $\rho f_y \geq (b_o\lambda\sqrt{f'_c}\ \alpha_s)/\ 80d$ in psi $(b_o\lambda\sqrt{f'_c}\ \alpha_s)/\ 960d$ in MPa (6)

For typical variations in c_1/d and f'_c, the ρ values for Grade 60 (414 MPa) reinforcement are given in Table 2.

Table 2: Variation in ρ with c_1/d and f'_c

f'_c, psi (MPa)	c_1/d		
	2	3	4
3,000 (20.7)	0.0055	0.0073	0.0091
4,000 (27.6)	0.0063	0.0084	0.0105
5,000 (34.0)	0.0071	0.0094	0.0118

ρf_y in Eq. 4 is made extendable to edge and corner columns by expressing it in terms of α_s where α_s is given in 22.6.5.3 of ACI 318-14. Use of a ρf_y value is recommended rather than a ρ value because designs using reinforcement yield strengths greater than 60ksi (414 MPa) are becoming increasingly common.

Shown in Fig.5 is the correlation between ρ and V_{test} for the same test data as that shown in Fig.3. The reinforcement ratio was calculated within 1.5h on either side of the column or loaded area. On the vertical axis, $V_{ACI} = V_c = 4\lambda\sqrt{f'_c}(b_o d)$ in psi ($0.33\lambda\sqrt{f'_c}(b_o d)$ in MPa) and k_v is a correction factor for the slab depth effect discussed later.

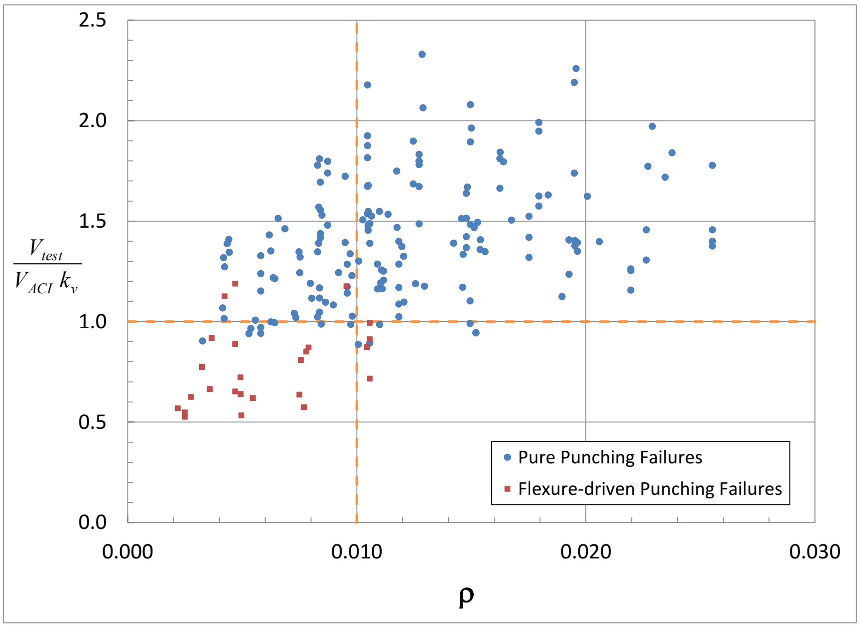

Figure 5: Correlation between V_{test} and ρ for connections transferring shear only.

Figure 6 shows a figure similar to Fig. 5 except that the calculated shear capacity has been taken as the lesser of $V_{ACI} = V_c = 4\lambda\sqrt{f'_c}(b_o d)$ in psi ($0.33\lambda\sqrt{f'_c}(b_o d)$ in MPa), and that defined by Eq. 2. Figure 6 shows that the non-conservatism of the ACI 318-14 design provisions for slabs with flexure-driven punching failures can be eliminated for most of the tests using the proposed recommendations.

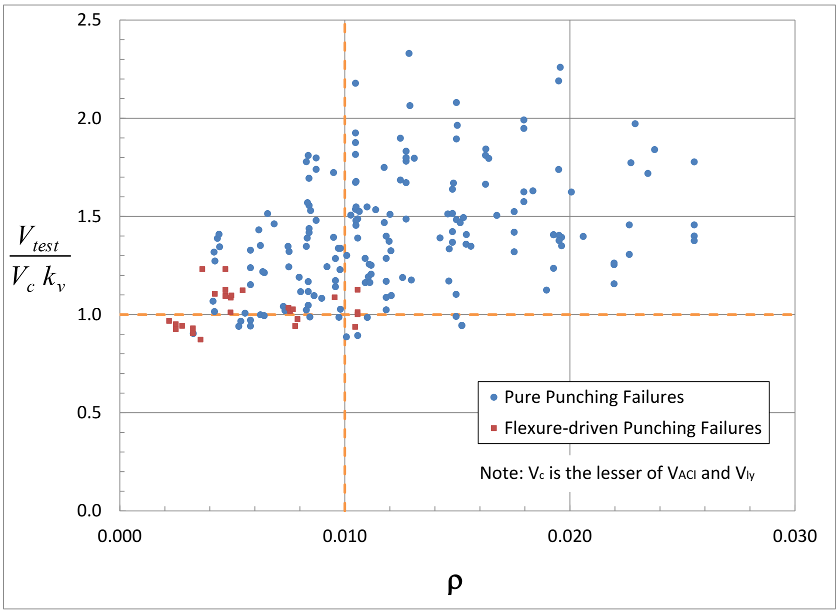

Figure 6: *Correlation between V_{test} and ρ for connections transferring shear only using proposed recommendations.*

4.3 Reinforcement requirements for slabs with shear reinforcement

For slabs with shear reinforcement, ACI 318-14 specifies that $v_n = v_c/2 + v_s$, when stirrups are used and $v_n = 3v_c/4 + v_s$ when headed studs are used. For simplicity in design, it is recommended that the same ρ limit be used regardless of the type of shear reinforcement. Per Table 22.6.6.2 of ACI 318-14, the maximum value for v_n when stirrups are used is $6\sqrt{f'_c}$ psi ($0.50\sqrt{f'_c}$ MPa) so that for consistency with requirements for slabs without shear reinforcement, the minimum required ρf_y for slabs with shear reinforcement could be as much as 50% greater than that for slabs without shear reinforcement. Because v_n is rarely provided up to the maximum $6\sqrt{f'_c}$ psi ($0.50\sqrt{f'_c}$ MPa) limit, a 4/3 multiplication is proposed for the minimum flexural reinforcement ρf_y for slabs with shear reinforcement.

5 Effect of slab depth on punching shear strength

Numerous investigators have reported a decrease in punching shear strength with increasing slab depth. See for instance Mitchell et al. (2005). A statistical study (Dönmez and Bažant, 2016) using the information in the NEES database (Ospina et al., 2011) has demonstrated the dependence of punching strength as a function of $1/\sqrt{(1+d/d_o)}$, where d_o is a constant. Selected test results from the NEES database indicate that this depth effect factor can be applied for values of d greater than 10 in. (250 mm). Corrections to the basic expressions for punching shear strength for depth effects are currently included in the Canadian, EC2, and Japanese building codes.

Shown in Fig. 7 are measured punching strengths for laboratory tests where investigators made systematic changes to the slab depth of their test specimens. Reinforcement ratios varied from 0.33 to 1.15%. Details of the data for the investigations shown in Fig. 7 can be found in the NEES database. References are listed in Ospina and Hawkins (2013). Measured strengths for tests decrease with increasing slab depth and are less than the ACI 318-14 predicted strengths for effective depths greater than 10 in. (250 mm). The influence of the slab reinforcement ratio when combined with the slab depth effect is rather inconclusive for the available test data. Figure 7 includes alternative depth effect factors that produce reasonable strength predictions for the available test data. The so-called "smooth" formulation ($1.4/\sqrt{(1+d/10)}$ (d in inches)) ($1.4/\sqrt{(1+d/250)}$ (d in mm)) provides a continuous correction across the full range of d values. This factor is to be applied for depths greater than 10 in. (250 mm). A reasonable lower bound can be also approximated by a depth effect relationship of $2.5/d^{2/5}$ (d in inches) ($9/d^{2/5}$, d in mm). A third factor of the form $3/\sqrt{d}$ (d in inches) ($15/\sqrt{d}$) (d in mm)) produces the most conservative predictions. It is recommended that the values of Table 22.6.5.2 of ACI 318-14 be modified by $1.4/\sqrt{(1+d/10)}$ (d in inches) to recognize the depth effect on punching strength. It is worth noting that the single test result by Guandalini et al. (2009) shown below the proposed design curve corresponds to a slab with a very low reinforcement ratio. Calculations (not reported herein) show that this slab would have punched at a load level close to what the proposed depth effect formulation predicts, had the minimum amount of flexural reinforcement proposed herein been provided to this test slab.

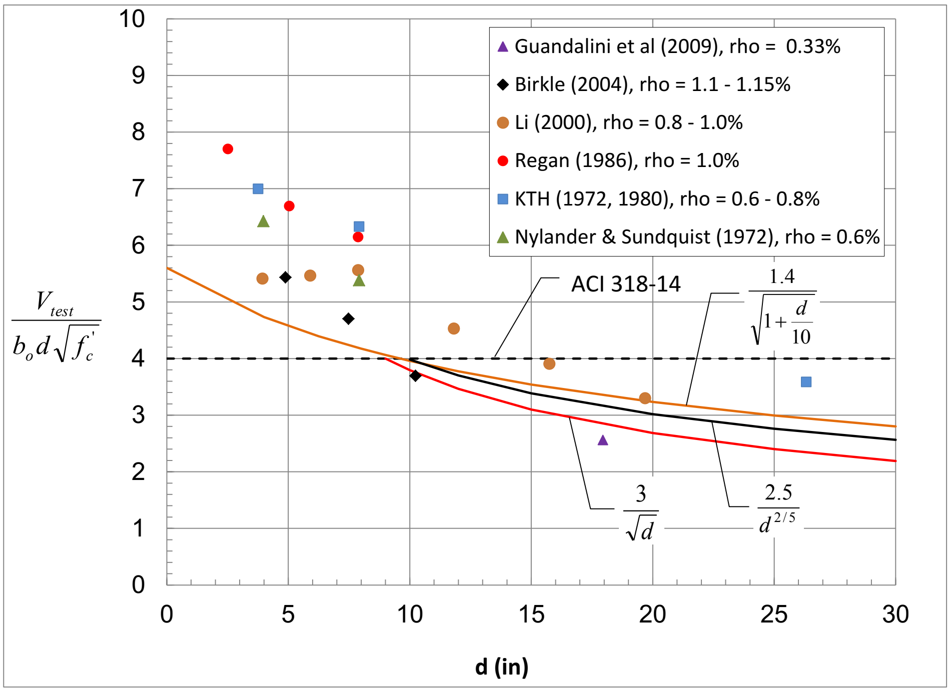

Figure 7: Variation in punching shear strength with slab effective depth. (1 in. = 25.4 mm)

6 Conclusions and recommendations

Two limitations to the existing punching shear provisions of ACI 318-14 should be recognized:

1. There can be flexure-driven punching shear failures resulting from yielding of the slab flexural reinforcement in the immediate vicinity of a column or concentrated load area. Those failures are similar in appearance to "pure" punching shear failures and the additional ductility resulting from a flexure-driven, as compared to a "pure" punching shear failure, can be small.
2. For punching shear failures, there is a decrease in the nominal shear strength with increasing slab depth.

To address the flexure-driven punching shear issue, ρf_y for the slab flexural tension reinforcement within $1.5h$ of the perimeter of the column or concentrated load area should be equal to or greater than $(b_o\lambda\sqrt{f'_c}\,\alpha_s)/80d$ psi $((b_o\lambda\sqrt{f'_c}\,\alpha_s)/960d$ MPa) for slabs without shear reinforcement and 1/3 greater for slabs with shear reinforcement.

To address the slab depth effect, the existing two-way nominal shear strength values of Table 22.6.5.2 of ACI 318-14 should be limited to slabs with effective depths of 10 in. (250 mm) or less. For greater depths, the values should be reduced in proportion to $1.4/\sqrt{(1+d/10}$ (d in inches) $(1.4/\sqrt{(1+d/250)}$ (d in mm).

7 Notation

The notation of this paper is that of ACI 318-14 with the following additions:

k_v = Slab depth effect factor = $1.4/\sqrt{(1+d/10)}$ (d in inches) $(1.4/\sqrt{(1+d/250)}$ (d in mm).

m = Nominal flexural strength of slab per unit width for the reinforcement within $1.5h$ of the column or concentrated load area

M_R = Calculated moment capacity of slab

M_T = Measured unbalanced moment in test

V_{flex} = Shear force for slab flexural failure

V_{ly} = Shear force for yielding of the slab flexural tension reinforcement within 1.5h of the column or concentrated load area

V_T = Measured shear force in test

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Appendix A

A.1 Overview

Summarized in this appendix are most available test data on the strength for simultaneous transfer of moment and shear to columns in flat plate concrete frames with flexural reinforcement ratios for the slab of about 1.1% or less in the vicinity of interior columns. The data are analyzed according to both the existing ACI 318-14 requirements and additional limitations proposed in this paper. With two exceptions, data for only interior columns are analyzed. The two exceptions are for slab-exterior column connections where moment was transferred to the column parallel to the edge. ACI 318-14 recognizes that unbalanced moment transfer for such columns is very similar to that for unbalanced moment transfer to interior columns. For exterior columns (moment transferred normal to the edge), the ACI 318-14 provisions effectively allow all unbalanced moment to be transferred by reinforcement within lines $1.5h$ on either side of the column when the shear being transferred along with that moment does not exceed $0.75\phi v_c$ for edge, and $0.50\phi v_c$ for corner columns. Therefore, the effect of unbalanced moment on shear strength does not have to be considered, but can be considered, if an increase in the amount of slab reinforcement is acceptable. By contrast, for interior slab-column connections, there is little ability to redistribute the unbalanced moment and it is nearly always necessary to consider the effect of that moment on punching shear strength.

Given at the end of Appendix A are relevant references for the column to slab connections analyzed here. The analysis is for 48 subassemblies and 12 slab systems where slab top reinforcement ratios were 1.1% and less, columns were square, and there was no shear reinforcement in the slabs. Omitted from the data are results for most tests with slab reinforcement ratios higher than 1%, lightweight concrete, rectangular columns, column capitals, tests dealing with any strengthening methods, and any results for subassemblies with slab overall depths less than 4.5 in. (115 mm). Those omissions are deliberate as they are not directly relevant to understanding the validity of the code limitation discussed here. Further, when slab overall depths are less than 4.5 in. (115 mm) slight variations in reinforcement depths can significantly affect results. The analysis of the data is in two parts. Part I is for tests on subassembly specimens with a single central column. Part II is for the interior column of slab system tests. For the slab system tests the slab thickness requirement of 4.5 in. (115 mm) or more was not imposed because all but one of the tests had lesser thickness.

In Part I the properties of the test specimens are provided in Table A1.1. Column 1 lists the reference where the data for the specimen listed in column 2 are found. Column 3 lists the specimen type. Columns 4 and 5 list the reported concrete strength at time of test and the measured yield strength for the slab top flexural reinforcement. Column 6 lists three quantities: (1) the reinforcement ratio for the top flexural bars in the slab based on the total width of the slab; (2) the same ratio for the top bars within lines $1.5h$ on either side of the column; and (3) the same ratio for the bottom bars within lines $1.5h$ on either side of the column. In general, the first ratio is consistent with the value provided in the reference. To derive the second ratio, an examination was made of the actual number of bars within the width $c_2 + 3h$ and the ratio calculated accordingly unless the spacing between bars was such that the distance between the line at $1.5h$ and the next bar was less than half the bar spacing. In that case, the number of bars was taken as one greater than the number within the width

$c_2 + 3h$. The reinforcement ratio for the bottom bars was calculated in the same manner as for the top bars. Column 7 lists two quantities, the first being the average effective depth for the top bars, and the second the overall depth of the slab. While the average effective depth is used for shear and flexural evaluations and the reinforcement ratios reported in column 6, a slightly different quantity, the actual depth for the slab reinforcement in the direction of moment transfer for both top and bottom bars was used for evaluations of the fraction of the unbalanced moment transferred by the slab flexural reinforcement. However, some researchers omitted exact details for bar size, spacing, and yield stress, and therefore calculations were based on the d values in column 7. Column 8 lists the column side dimension. All specimens had square columns. Column 9 lists the slab outside to outside dimension with the dimension in the direction of moment transfer listed first. Column 10 lists the type of loading to which the specimen was subjected.

Table A1.2 contains results of analyses in accordance with ACI 318-14 and the proposed code change. Column 1 lists the specimen name. Columns 2 through 5 list, respectively, the shear, V_T, acting on the connection at failure; the unbalanced moment, M_T, acting on the connection at failure; the nominal shear strength, $V_0 = 4\sqrt{f'_c}*b_o*d$ according to ACI 318-14 for direct shear transfer only; and the corresponding quantity, $M_0 = 4\sqrt{f'_c}*J_c/(\gamma_v*(c_1/2 + d/2))$, for transfer of moment only. Column 6 lists the sum of the ratios for measured to calculated shear and moment strengths. For values of that sum greater than 1.0 a punching shear failure is predicted. Listed in column 7 is the amount of unbalanced moment calculated by ACI 318-14 transferred to the column by the flexural reinforcement in the slab within lines *1.5h* on either side of the column. Listed in column 8 is the moment M_R that can be transferred by the top and bottom flexural reinforcement in the slab within the lines *1.5h* on either side of the column. The ratio in column 9 is $\gamma_f M_T$ divided by M_R. For values greater than 1.0, a flexure-driven punching failure resulting from inadequate moment transfer reinforcement is predicted. Column 10 lists the theoretical nominal flexural strength for the connection for shear transfer only. That prediction is 8m where m is the average nominal moment per unit width provided by the top flexural reinforcement in the slab within lines *1.5h* on either side of the column for the length of the column perimeter. Column 11 lists the sum of the ratios of the measured to calculated unbalanced moment strength plus the ratio of the measured shear to the shear 8m. If that ratio is greater than 1.0 a flexure-driven punching failure is predicted. Column 12 lists the failure mode as apparent from measured load-deflection results. If the ductility, defined in the manner reported by Hawkins et al. (1989) exceeded 1.8, the failure mode of column 12 is reported as F/P, a flexure-driven punching failure. If the ductility is less than 1.8, the failure mode is defined as a punching failure, P.

In Part II, the properties of the test frames are listed in Table A2.1 and the test results are listed in Table A2.2. The columns in Tables 2.1 and 2.2 have the same meaning as those for subassembly tests in Part I. Results for 12 interior columns that were part of test frames are reported in Part II. In all 12 frame tests, punching failures occurred first at the interior, rather than the exterior, column connections.

A.2 ACI 318-14 strength predictions and ACI 318-14 strength predictions amended as proposed

In Tables A1.2 and A2.2, the highest of the three ratios (columns 6, 9, and 11 in Table A1.2 and columns 5, 8, and 10 in Table A2.2) is shown in bold. An examination of those results indicates the appropriateness or non-appropriateness of this change proposal for the moment and shear transfer situation. Comparison of the ratios for the two columns 6 and 9 in Table A1.2 and 5 and 8 in Table A2.2 indicates the same situation for the existing code provisions for moment and shear transfer.

For Table A1.2, for the ACI 318-14 provisions, there are 25 results controlled by the punching shear strength provision of column 6 and 23 results by the flexural reinforcement strength limitation of column 9. The mean of the results where column 6 prevails is 1.23 and individual values range from 0.79 to 2.02. The mean of the results where column 9 prevails is 1.29 and individual results range from 0.99 to 2.17. For analysis using the ACI 318-14 provisions amended as proposed, there are 19 results where column 6 prevails, 18 results where column 9 prevails, and 11 results where column 11 prevails. The corresponding mean values and the range in individual values for columns 6, 9 and 11 are 1.35 (1.00-2.02), 1.38 (0.99-2.17) and 1.10 (0.95-1.22). More importantly, for the existing code there are seven results out of the 48 where the measured to calculated strength is less than 1.00. The lowest result is 0.79. With the proposed change only two results are less than 1.00 and the lowest result is 0.95. Adoption of the change improves the likelihood of the failure mode, as listed in column 12, agreeing with the calculated prediction.

The results in Part II for slab system tests are not as definitive as the results for subassembly tests. This is probably due to the difficulty of making such tests and the marked changes in properties with small changes in reinforcement positions within the slab. In Table A2.2, for the existing code provisions, there are six results for which column 6 (shear) controls (mean 1.12, range 0.91-1.36) and six results for which column 9 (moment transfer reinforcement) controls (mean 1.46, range 1.17-1.72). When the limitation proposed here is used there are four results for which shear controls, six results for which moment transfer reinforcement controls, and two results where the flexure-driven punching failure controls. Thus, for the systems tested, the introduction of the flexure-driven criteria of the proposed code change has little effect when there is moment transfer to the columns in addition to shear. However, the result for G-S1 shows the marked change in the strength prediction for a low reinforcement ratio when there is shear transfer only. The introduction of the change provision also results in a better prediction of the mode of failure.

One issue apparent from the slab system tests is how to address the calculation of the flexure-driven shear capacity when there are different reinforcement ratios in orthogonal directions. This issue has not been addressed in the subassembly tests but is immediately apparent in slab system tests where there are different span lengths between columns in orthogonal directions. The best agreement with test data was obtained when the m value used to calculate V_F was taken as the m value for the reinforcement in the direction of the longer span. This finding was confirmed by using yield line analysis to find how V_F changed with changing orthogonal span lengths and therefore different amounts of slab flexural reinforcement in orthogonal directions.

Table A1.1 – Properties for subassemblage tests

Ref.	Spec. No.	Spec. type	f'_c psi	f_y ksi	ρ_t / ρ_{c+3ht}/ ρ_{c+3hb} %	d/h in.	Col. size in.	Slab size ft	Load type
1	1	SC	4825	68.4	0.76/0.98/0.33	3.98/4.8	10.8	12x12	C
	3	SC	4550	68.4	0.76/0.98/0.33	3.98/4.8	10.8	12x12	C
2	ND1C	SC	4290	60*	0.53/0.76/0	3.73/4.5	10.0	10x10	C
	ND4LL	SC	4680	60*	0.53/0.76/0	3.73/4.5	10.0	10x10	C
	ND5XL	SC	3500	60*	0.53/0.76/0	3.73/4.5	10.0	10x10	C
	ND6HR	SC	3810	60*	0.93/1.50/0	3.73/4.5	10.0	10x10	C
	ND7LR	SC	2730	60*	0.39/0.38/0	3.73/4.5	10.0	10x10	C
4	1C	SC	5130	61	0.49/1.00/0.50	3.73/4.5	9.84	9.8x9.8	C
6	SCO	SC	5700	76.1	0.86/0.90/0.51	3.63/4.5	10	9.5x6.5	C
7	HHC0.5	SC	10960	66.7	0.50/0.49/0.27	4.64/5.9	9.84	6.3x6.3	C
	HHC1.0	SC	10490	66.7	1.00/0.98/0.27	4.64/5.9	9.84	6.3x6.3	C
	NHC0.5	SC	5330	66.7	0.50/0.49/0.27	4.64/5.9	9.84	6.3x6.3	C
	NHC1.0	SC	5130	66.7	1.00/0.98/0.27	4.64/5.9	9.84	6.3x6.3	C
8	CO	SC	5600	66.0	0.46/0.54/0.23	5.12/6.0	10.0	9.5x9.5	C
9	6AH	SC	4550	68.5	0.56/0.56/0.25	4.75/6.0	12	7 x 7	M
	9.6AH	SC	4450	60.2	0.89/0.89/0.47	4.63/6.0	12	7 x 7	M
	6AL	SC	3300	68.5	0.56/0.56/0.25	4.75/6.0	12	7 x 7	M
	9.6AL	SC	4200	60.2	0.89/0.89/0.47	4.63/6.0	12	7 x 7	M
	7.3BH	SC	3220	68.5	0.68/0.68/0.25	3.25/4.5	12	7 x 7	M
	9.5BH	SC	2880	68.5	0.95/0.95/0.30	3.25/4.5	12	7 x 7	M
	7.3BL	SC	2630	68.5	0.68/0.68/0.25	3.25/4.5	12	7 x 7	M
	9.5BL	SC	2910	68.5	0.95/0.95/0.30	3.25/4.5	12	7 x 7	M
	6CH	SC	7600	68.5	0.56/0.56/0.25	4.75/6.0	12	7 x 7	M
	9.6CH	SC	8300	60.2	0.89/0.89/0.47	4.63/6.0	12	7 x 7	M
	6CL	SC	7190	68.5	0.56/0.56/0.25	4.75/6.0	12	7 x 7	M
	6FLI	SC	3760	68.5	0.56/0.84/0.25	4.75/6.0	12	7 x 7	M
10	SM0.5	SC	5330	69.0	0.5/0.5/0.35	4.75/6.0	12	6 x 6	M
	SM1.0	SC	4840	69.0	1.0/1.10/0.35	4.75/6.0	12	6 x 6	M
11	1	SC	5075	65.5	1.1/1.0/0.44	4.6/5.9	9.8	6 x 6	M
12	S2	SC	3400	67.1	0.9/0.9/0.49	4.63/6.0	12	13 x 7	C
	S3	SC	3200	66.0	0.57/0.57/0.40	4.75/6.0	12	13 x 7	C
	S7	SC	3840	67.1	0.9/0.9/0.49	4.63/6.0	12	13 x 7	C
	S8	SC	4470	66.0	0.57/0.57/0.40	4.75/6.0	12	13 x 7	C
	EL1	SEC	4620	67.1	0.81/0.81/0.40	5.13/6.5	12	13 x 4	C
	EL2	SEC	3520	65.0	1.07/1.07/0.49	5.0/7.0	16	13 x 4	C
13	HHS0.5	SC	10730	66.7	0.5/0.6/0.27	4.93/5.9	9.84	6 x 6	M
	HHS1.0	SC	10710	66.7	1.0/0.97/0.27	4.68/5.9	9.84	6 x 6	M
14	HLS0.5	SC	6270	66.7	0.5/0.6/0.27	4.93/5.9	9.84	6 x 6	M
	HLS1.0	SC	6190	66.7	1.0/0.97/0.27	4.68/5.9	9.84	6 x 6	M
	NHS1.0	SC	5250	66.7	1.0/0.97/0.27	4.68/5.9	9.84	6 x 6	M
	NHHS0.5	SC	4930	66.7	0.5/0.6/0.27	4.93/5.9	9.84	6 x 6	M
	NHHS1.0	SC	5110	66.7	1.0/0.97/0.27	4.68/5.9	9.84	6 x 6	M
16	C-02	SIC	4480	65.8	0.96/0.97/0.48	3.23/4.5	12	9 x 9	C
17	SW1	SC	5080	75.4	1.09/1.09/0.56	3.54/4.72	7.87	5 x 5	C
	SW5	SC	6670	75.4	1.09/1.09/0.56	3.54/4.72	7.87	5 x 5	C
18	L0.5	SC	3710	68	0.5/0.5/0	5.00/6.0	16	14 x 14	C
	LG0.5	SC	4820	66	0.5/0.5/0	5.00/6.0	16	14 x 14	C
	LG1.0	SC	4000	61	1.0/1.0/0	5.00/6.0	16	14 x 14	C

Notes for Table A1.1: **Specimen type**: SC = slab-interior column; SIC = slab-interior concentrated load; SEC = slab-edge column with moment transferred parallel to edge. **Loading**: C = cyclic**;** M = monotonic. G = gravity; *Grade 400 MPa bars. Actual yield stress not reported.
(1,000 psi = 1 ksi = 6.895 MPa; 1 inch = 25.4 mm; 1 ft = 0.305 m)

Table A1.2: Results for subassemblage tests

Spec. No	V_T kips	M_T kip-in.	V_o kips	M_0 kip-in.	$V_T/V_0+M_T/M_0$	$\gamma_f M_T$ kip-in.	M_R kip-in	$\gamma_F M_T/M_R$	V_F kips	$V_T/V_F+M_T/M_0$	Failure mode
1	26.0	557	65.4	821	1.08	334	343	0.97	81.7	1.00	P
3	16.3	865	63.5	797	1.35	519	343	1.51	81.7	1.28	F/P
ND1C	13.28	347	53.7	626	0.80	208	142	1.46	31.8	0.97	F/P
ND4LL	15.94	379	56.1	654	0.86	221	142	1.56	31.8	1.08	F/P
ND5XL	22.75	275	48.5	566	0.97	165	142	1.16	31.8	1.21	P
ND6HR	14.73	492	50.6	590	1.12	295	256	1.15	61.8	1.07	P
ND7LR	11.02	227	42.8	499	0.72	136	103	1.27	24.4	0.91	F/P
1C	8.5	516	58.1	671	0.92	310	290	1.07	49.1	0.94	P
SCO	16.0	543	59.7	707	1.04	326	314	1.04	68.1	1.00	F/P
HHC0.5	28.1	1190	118	1393	1.09	714	329	2.17	74	1.23	F
HHC1.0	28.1	1442	115	1363	1.30	865	505	1.72	148	1.25	F/P
NHC0.5	28.1	889	84.3	971	1.25	533	329	1.62	74	1.29	F/P
NHC1.0	28.1	1126	78.4	953	1.54	676	505	1.34	148	1.37	P
CO	28.1	912	92.7	1203	1.06	547	370	1.48	71.6	1.15	P
6AH	38.1	800	85.9	1223	1.10	480	405	1.19	66.5	1.22	F/P
9.6AH	42.0	865	82.4	1153	1.25	519	604	0.86	88.2	1.23	P
6AL	54.8	289	76.1	1032	1.04	173	399	0.43	66.5	1.10	P
9.6AL	57.8	306	80.3	1133	0.99	184	602	0.30	88.2	0.93	P
7.3BH	17.9	345	44.8	575	1.00	207	190	1.09	36.4	1.10	F/P
9.5BH	21.2	402	42.4	551	1.25	330	265	1.25	48.6	1.17	F/P
7.3BL	29.2	113	41.1	514	0.94	68	184	0.37	36.0	1.03	F/P
9.5BL	31.9	147	42.5	544	1.03	88	265	0.33	48.6	0.93	P
6CH	41.9	842	110.	1588	0.90	505	411	1.23	67.7	1.15	F/P
9.6CH	48.9	1001	111.	1589	1.07	601	622	0.97	88.2	1.18	F/P
6CL	61.4	326	108.	1552	0.79	196	411	0.48	67.6	1.12	F/P
6FLI	51.0	240	78.5	1091	0.87	144	520	0.28	68.6	0.96	P
SM0.5	29.0	888	92.9	1297	0.99	533	359	1.48	60.3	1.16	F/P
SM1.0	29.0	1128	88.4	1235	1.24	677	601	1.13	117	1.16	P
1	33.7	1151	77.9	969	1.62	691	612	1.13	118	1.47	P
S2	32.0	778	71.1	1010	1.22	467	629	0.74	94.2	1.11	P
S3	31.2	475	72.6	1033	0.89	285	402	0.71	63.9	0.95	F/P
S7	60.8	376	76	1075	1.15	226	633	0.21	95.4	0.99	P
S8	52.8	289	85.2	1204	0.86	173	416	0.66	65.0	1.05	F/P
EL1	17.0	779	65.4	1053	1.00	450	518	0.87	80.6	0.95	P
EL2	18.6	1159	77.5	1588	0.97	673	681	0.99	114	0.89	P
HHS0.5	45.0	1044	122.	1513	1.06	626	374	1.67	72.0	1.32	F/P
HHS1.0	59.0	1173	114.	1413	1.35	704	575	1.22	111	1.36	F/P
HLS0.5	59.9	393	92.2	1156	0.99	236	366	0.64	71	1.18	F/P
HLS1.0	91.8	467	87.4	1061	1.49	280	557	0.50	108	1.29	P
NHS1.0	36.8	1040	80.0	990	1.51	624	546	1.14	106	1.40	P
NHHS0.5	36.9	865	82	1030	1.29	519	360	1.44	69.8	1.37	F/P
NHHS1.0	56.3	1028	78.2	979	1.77	617	544	1.13	106	1.59	F/P
C-02	20.2	394	52.7	677	0.96	236	237	1.00	49.4	0.99	P
SW1	24.73	608	46.0	448	1.90	365	314	1.16	75.9	1.69	P
SW5	36.0	689	52.8	514	2.02	413	319	1.29	77.2	1.81	P
L0.5	23.5	1094	103	1818	0.83	656	414	1.58	64.9	0.96	F/P
LG0.5*	26.8	1028	117	2071	0.73	617	404	1.53	63.3	0.92	F/P
LG1.0*	24.1	1360	106	1885	0.95	816	743	1.10	116.5	0.93	F/P

Notes for Table A1.2: Failure mode: P =punching; F/P = flexure/punch. * After lateral loading specimens were loaded to failure under gravity load only. LG0.5 punched at 72.8 kips (1.12 V_F) after some yielding. LG1.0 punched at 89.9 kips (0.77 V_F) after little yielding.
(1 kip = 4.448 kN; 1 kip-in = 0.113 kN-m)

A.3 Design examples

The practicality of the proposed code changes was examined using the designs for two reinforced concrete structures completed by a major consulting firm. One structure had 28 by 28 inch (711 by 711mm) square columns, 30 ft (9.1 m) spans in orthogonal directions, and a 10 in. (250 mm) thick slab. The factored reaction for the typical interior column was 240 kips (1068 kN) and no moment transfer. The required top reinforcement was 25#6 bars for each direction for the column strip negative moment reinforcement. The second structure had 12 by 12 inch (305 by 305 mm) columns, 24 ft (7.3 m) spans in orthogonal directions, and an 8 in. (203 mm) thick slab. The factored reaction for the typical interior column was 135 kips (600 kN) and no moment transfer. The required top reinforcement was 16#6 bars for each direction for the column strip negative moment reinforcement along with shear reinforcement consisting of four legs of #3bars@3 in. (76 mm) on all sides of the column. For both designs the specified concrete strength was 4,000 psi (27.6 MPa) along with Grade 60 (414 MPa) reinforcement. In addition to slab dead load, there was a superimposed dead load of 30 psf (1.44 MPa) and a live load of 50 psf (2.39 MPa). The number of bars and their sizes for the column strip, and middle strip, for a typical interior span, an end span with the exterior column transferring moment normal to the edge, and a corner (end span in orthogonal directions) were specified in accordance with ACI 318-14 requirements. The impact of the recommended code changes on the design of the slab reinforcement in the vicinity of a typical interior column, edge column, and corner column was examined.

For both building designs, while the column strip reinforcement was nominally less than 1% for the edge and corner column designs, no changes were needed to the prior designs. The existing code requirement to concentrate column strip bars within lines *1.5h* on either side of the column to transfer the fraction of the unbalanced moment not transferred by shear resulted in reinforcement ratios in that region greater than 1% and therefore there was no effect of the proposed flexure-driven punching shear requirement. For the interior column for the 30 ft (9.1 m) span, a slight concentration of the slab negative moment column strip reinforcement within the lines *1.5h* on either side of the column was adequate to satisfy the proposed requirement. No additional column strip reinforcement was needed. For the interior span for the 24 ft (7.3 m) span, shear reinforcement was specified in the original design to meet existing code requirements. Even with the enhanced shear strength provided by that shear reinforcement, some concentration of top bars within lines *1.5h* on either side of the column was adequate to ensure that the proposed limit was satisfied. Again, no additional column strip reinforcement was needed.

Part II: Frame data

Robertson and Durrani (1992) and Durrani et al. (1995): Tests of interior column of two-bay (9.5 ft (2.9 m) c.t.c. of columns); 6.5 ft (2.0 m) wide slab.

Sherif and Dilger (2000): Full-scale test of frame consisting of one end frame with and edge column and one interior column with an overhang extending to the line of contraflexure. End frame 16 ft-5 in. (5.0 m) c.t.c. of columns and interior overhang of 8 ft-3 in. (2.5 m). Slab width of 16 ft-5 in (5.0 m).

Rha et al. (2014): Half scale tests of a 2 bay x 2 bay frame (one interior column, four edge columns and two corner columns). Bays of 9 ft (2.75 m) in lateral load direction and 5 ft-5 in. (1.65 m) in transverse direction.

Table A2.1: Specimen properties

Ref.	Spec. No.	f'_c psi	f_y ksi	ρ_t / ρ_{c+3ht} / ρ_{c+3hb} %	d/h in.	Col. size in. x in.	Load type
3	A	4790	72.6	0.49/0.83/0.38	3.66/4.5	10	C
	B	4460	76.1	0.49/0.83/0.38	3.66/4.5	10	C
	C	4670	76.1	0.49/0.83/0.38	3.66/4.5	10	C
5	DNY1	5120	54	0.55/0.74/0	3.81/4.5	10	C
	DNY2	3730	54	0.55/0.74/0	3.81/4.5	10	C
	DNY3	3570	54	0.55/0.74/0	3.81/4.5	10	C
15	S1	4060	64.4	1.29/1.29/0.20	5.03/5.9	9.8	M
19	G-S1	3770	59.5	0.74/0.72/0.39	2.57/3.5	9.84	G
	G-S2	3770	59.5	0.98/1.26/0.39	2.57/3.5	9.84	G
	LM-S2	3770	59.5	0.74/1.26/0.42	2.57/3.5	9.84	M
	LM-S3	3770	59.5	0.98/1.26/1.05	2.57/3.5	9.84	M
	LC-S2	3770	59.5	0.74/1.26/0.42	2.57/3.5	9.84	C

Note: 1,000 psi = 1 ksi = 6.895 MPa; 1 inch = 25.4 mm; 1 ft = 0.305 m

Table A2.2: Test results and strength calculations for the interior column of the frame

Spec. No.	V_T kips	M_T kip-in	V_o kips	M_0 kip-in	$V_T/V_0 + M_T/M_0$	$\gamma_f M_T$ kip-in	M_R kip-in	$\gamma_F M_T/M_R$	V_F kips	$V_T/V_F + M_T/M_0$	Failure mode
A	10	623	55.6	639	1.15	374	251	1.49	60.2	1.04	F
B	19.3	366	53.4	616	0.96	220	263	0.84	63.0	0.90	P
C	28.8	240	27.1	630	0.91	144	263	0.55	60.4	0.85	P
DNY1	15.4	418	60.2	706	0.85	251	149	1.68	43.3	0.95	F
DNY2	19.2	296	51.4	602	0.87	178	149	1.19	43.3	0.94	P
DNY3	11.9	390	50.3	590	0.95	257	149	1.72	43.3	0.99	F
S1	89.7	3.54	72	967	1.25	-	-	-	164.8	0.54	P
G-S1	34.34	0	31.36	0	1.10				23.4	1.47	F/P
G-S2	35.88	0	31.36	0	1.14	-	-	-	35.1	1.02	P
LMS2	11.0	332	31.36	327	1.37	199.2	127.8	1.55	23.4	1.49	F/P
LMS3	13.8	300	31.36	327	1.36	180	176	1.02	30.9	1.37	P
LC-S2	10.9	255	31.36	327	1.13	153	127.8	1.19	23.4	1.13	P

Note: 1 kip = 4.448 kN; 1 kip-in = 0.113 kN-m

Appendix A – References

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Design examples

Structure 1: 28 in. square columns and 30 ft spans; d = 8.5 in; $v_c = 4\sqrt{f'_c} = 253$ psi

Interior column: $b_o = (28 + 8.5)*4 = 146$ in.

$\phi V_n = \phi V_c = \phi\ 4\sqrt{f'_c}\ b_o\ d = 0.75*0.253*146*8.5 = 236$ kips

$V_u = 240 - 2.6 = 237.4$ kips $\approx \phi V_n$

Slab flexural reinforcement requirements within 1.5*h* of column perimeter

Required $\boldsymbol{\rho f_y} = (b_o\sqrt{\boldsymbol{f'_c}}\ \boldsymbol{\alpha_s})/80d$

Required $\boldsymbol{\rho}$ = 146*63*40/80*8.5*60,000 = 0.0090

For uniform top bar spacing over column strip width s = (span/2)/no. of bars

s = 15 *12/25 = 7.2 in. and existing $\boldsymbol{\rho}$ = 0.44/(7.2*8.5) = 0.0072

Reduce bar spacing within (c + 3h) to 5.75 in.

No of bars in (c + 3h) = (28 + 3*10)/5.75 = 10.1 bars. Say 10 bars.

$\boldsymbol{\rho}$ = 0.44/5.75*8.5 = 0.0090 OK

Bar spacing outside (c + 3h) = (15*12 - 58)/(25 - 10) = 8.1 in. OK

No extra reinforcement is needed. Only requirement is concentration of about half of required column strip reinforcement within lines 15 in. on either side of column.

Structure 2: 12 in. square columns and 24 ft spans; d = 6.5 in; $v_c = 4\sqrt{f'_c} = 253$ psi

Interior column: $b_o = (12 + 6.5)*4 = 74$ in.

$\phi V_n = \phi V_c = \phi\ 4\sqrt{f'_c}\ b_o\ d = 0.75*0.253*74*6.5 = 91.3$ kips

V_u = 135 kips and therefore shear reinforcement is required

Required V_s = (135 – 91.3/2)/0.75 = 119 kips

For four leg #3 stirrups at 3 in. spacing on all four sides

$V_s = n\ A_s f_y\ d/s$ = 16*0.11*60* 6.5/3 = 229 kips OK

Slab flexural reinforcement requirements within 1.5*h* of column perimeter

Required $\boldsymbol{\rho f_y} = (b_o\sqrt{\boldsymbol{f'_c}}\ \boldsymbol{\alpha_s})/60d$

Required $\boldsymbol{\rho}$ = 74*63*40/60*6.5*60,000 = 0.0080

For uniform top bar spacing over column strip width s = (span/2)/no. of bars

s = 12 *12/16 = 9 in. and existing $\boldsymbol{\rho}$ = 0.44/(9*6.5) = 0.0075

Reduce bar spacing within (c + 3h) to 7.5 in.

No of bars in (c + 3h) = (12 + 3*8)/7.5 = 4.8 bars. Try 5 bars.

$\boldsymbol{\rho}$ =0.44/(6.5*7.5) = 0.0090 OK

Bar spacing outside (c + 3h) = (12*12 - 36)/(16 - 5) = 9.8 in. < 16 in. OK

No extra reinforcement is needed. Only requirement is concentration of about one third of required column strip reinforcement within lines 12 in. on either side of column.