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Design of Deep Pile Caps by Strut-and-Tie Models


by Perry Adebar and Luke (Zongyu) Zhou

Comparisons with results from 48 pile cap tests demonstrate that the one-way shear design provisions of the present ACI Building Code are excessively conservative for deep pile caps, and that the traditional flexural design procedures for beams and two-way slabs are unconservative for pile caps. Flexural design can best be accomplished using a simple strut-and-tie model, and test results demonstrate that the longitudinal reinforcement should be concentrated over the piles as suggested by strut-and-tie models. A simple shear design procedure is proposed in which maximum bearing stress is considered the best indicator of "shear strength" for deep pile caps. The maximum bearing stress that can be applied without causing splitting of compression struts within pile caps depends on the amount of confinement, as well as the aspect ratio (height-to- width) of compression struts. The influence of confinement is more gradual than suggested by the ACI Code bearing strength provisions.

Keywords: building codes; caps (supports); deep beams; footings; piles; reinforced concrete; shear strength; structural design; strut-and-tie models; tests.

The ACI Building Code procedure for the shear design of footings supported on piles (pile caps) is the same sectional approach used for footings supported on soil and for two-way slabs. The procedure involves determining the section thickness that gives a concrete contribution V_c greater than the shear force applied on the code- defined critical section. While this approach is reasonable for slender footings supported on numerous piles, it is not appropriate for deep pile caps.

A change recently introduced in the ACI Building Code[1] means that the critical section for one-way shear in deep pile caps is now at the column face rather than d from the column face. This relatively small change in location of the critical section has resulted in a very significant increase in the required depth of many deep pile caps. The fact that a small change in location of the critical section has such a large consequence is a demonstration that a sectional approach is not appropriate in this case. It is also important to note that the drastic increase in the ACI Code shear requirements for deep pile caps implies that either the present method is overly conservative or that previously designed deep pile caps may be unsafe.

As the ACI Code shear design procedures are not appropriate for deep pile caps (they were not developed for that purpose), the CRSI Handbook[2] suggests an alternate one-way shear design procedure when the center of the nearest pile is within d from the column face, and an alternate two-way shear design procedure when the center of the nearest pile is within $d/2$ from the column face. The CRSI Handbook alternate procedures involve a critical section along the column face for both one-way and two-way shear, as well as modified expressions for the concrete contribution.

Another approach for deep pile caps is to use strut-and-tie models[3,4,5] that consider the complete flow of forces rather than the forces at any one particular section. The internal load path in cracked reinforced concrete is approximated by an idealized truss, where zones of concrete with primarily unidirectional compressive stresses are modeled by compression struts, tension ties are used to model the principal reinforcement, and the areas of concrete where strut and ties meet (referred to as nodal zones) are analogous to joints of a truss. While the concept of using a truss analogy for the flexural design of deep pile caps (i.e., determining the required amount of longitudinal reinforcement) is well known,[6,7,8] a sectional approach has invariably been used for the shear design of pile caps.

Unlike traditional design procedures, strut-and-tie models do not separate flexural and shear design; however, it may be said that the "shear design" of deep members using strut-and-tie models involves limiting the concrete stresses to insure that the tension tie reinforcement yields prior to a concrete shear failure. If sufficient distributed reinforcement is provided to insure crack control, thereby allowing internal redistribution of stresses after cracking, the compressive stresses in the concrete struts should be limited depending on the biaxial strains.[4] On the other hand, if little or no reinforcement is provided for crack control, the concrete tensile stresses should be limited to avoid diagonal cracking of compression struts.[5] In pile caps it is usually not practical to provide sufficient distributed (horizontal and vertical)

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reinforcement to insure crack control; therefore, diagonal cracking of the compression struts should be avoided. Adebar and Zhou[9] have recently developed bearing stress limits to avoid transverse splitting in concrete compression struts confined by plain concrete, similar to the situation that occurs in pile caps. Utilizing these concrete stress limits, strut-and-tie models can be used for both "flexural design" and "shear design" of deep pile caps.

In this paper the methods commonly used in North America for the design of deep pile caps are briefly reviewed. This includes the ACI Building Code with and without the recent modifications, as well as the method suggested in the CRSI Handbook. A shear design procedure for deep pile caps based on the strut-and-tie model concept is presented, and results from 48 deep pile cap tests are reviewed and compared with predictions from the different design methods.

RESEARCH SIGNIFICANCE

Deep pile caps are important structural elements that are not adequately covered by the ACI Building Code. Many pile caps are designed by design aids with rule-of-thumb procedures and what are hoped to be conservative allowable stresses, but considerable disparity exists between the various procedures.

The information presented in this paper should prove useful to the organizations who publish design aids for deep pile caps and practicing engineers who must design appropriate pile cap designs.

DESIGN METHODS

ACI Building Code

The ACI Building Code (ACI-318) does not contain any provisions specifically for deep pile caps. Thus, designs are based on the procedure for slender footings that can be divided into three separate steps: 1) shear design, which involves calculating the minimum pile cap depth so that the concrete contribution to shear resistance is greater than the shear applied on the code-defined critical sections for shear; 2) flexural design, in which the usual assumptions for reinforced concrete beams are used to determine the required amount of longitudinal reinforcement at the critical section for flexure; and 3) a check of the bearing stress at the base of the column and at the top of the piles.

The special provisions for the shear design of slabs and footings (Section 11.12) requires that designers consider both one-way and two-way shear. In the 1977 and earlier editions of the ACI Code,[10] the special provisions for slabs and footings specifically stated that the critical section for one-way shear was located at a distance d from the face of the concentrated load or reaction area. In addition, Section 11.1 of the ACI Code stated that sections located less than a distance d from the face of support may be designed for the same shear as that computed at a distance d. The commentary to Section 11.1 warned that if the shear at sections between the support and a distance d differed radically from the shear at distance d, as occurs when a concentrated load is located close to the support, the critical section should be taken at the face of the support. Designers of pile caps could ignore this warning, however, since the specific statement in the code for slab and footings superseded the more general statement made in the commentary. In addition, a number of technical reports (e.g., Reference 11) described how the shear strength of deep members is much greater than the shear strength of slender members.

In the 1983 and subsequent editions of the ACI Code, the statement about the location of critical section for one-way shear was removed from the special shear provisions for slabs and footings, and the general statement about the critical section being at the face of the support when a concentrated load occurs within d from the support was moved from the commentary to the code. In addition, the commentary was modified to include a footing supported on piles as an example of when the critical section is commonly at the face of the support. The result is that designers of deep pile caps now have no choice but to take the critical section for one-way shear at the face of the column.

The ACI Building Code procedures for two-way shear have not been modified recently. The critical section remains at $d/2$ from the perimeter of the column regardless whether there is a concentrated load applied within the critical section. Section 15.5.3 states that any pile located inside the critical section is considered to produce no shear on the critical section and describes how to calculate the contribution from any pile that intercepts the critical section. The commentary on Section 15.5.3 contains a statement (since 1977) that when piles are located within the critical section, analysis for shear in deep flexural members, in accordance with Section 11.8, needs to be considered. Unfortunately, Section 11.8 of the ACI Code addresses only one-way shear in deep members, where the critical section is taken midway between the concentrated load and the support and the concrete contribution is increased due to deep beam action.

The ACI Building Code specifies that the critical section for moment in footings is at the face of concrete columns. The quantity of longitudinal reinforcement required at this critical section is determined by the usual procedures for reinforced concrete members, assuming plane sections remain plane and assuming that there is uniform flexural compression stresses across the entire width of the member. The designer is told to distribute the required longitudinal reinforcement uniformly across the footing (except that the short-direction reinforcement of rectangular footings must be somewhat more concentrated near the center).

According to the ACI Code, the maximum bearing strength of concrete is $0.85 f_c'$, except when the supporting surface area A_2 is wider on all sides than the loaded area A_1, the bearing strength is multiplied by $\sqrt{A_2/A_1}$ but not more than 2.

CRSI Handbook

The CRSI Handbook[2] makes use of the general design procedures in the ACI Building Code for the design of pile caps, with the exception of the shear design procedures for deep pile caps. When the center of the nearest pile is within d from the column face, the CRSI Handbook suggests that the one-way shear capacity should be investigated at the face of the column (similar to recent ACI Codes), but suggests that the concrete contribution should be significantly increased to account for deep beam action. The suggested relationship for one-way shear is

$$V_{c_{CRSI}} = \frac{d}{w} V_{c_{ACI}} \le 10\sqrt{f_c'}\, b\, d \qquad (1)$$

where w is the distance from the center of the nearest pile to the face of the column. The CRSI Handbook suggests that to include the effect of M/Vd for several piles at varying spans, the more complex ACI Code expression for V_c [Eq. (11-6)] should be used.

When the center of the nearest pile is within $d/2$, the CRSI Handbook suggests that the two-way shear capacity should also be investigated at the perimeter of the column face (this is different than the ACI code), and again, the concrete contribution should be increased to account for deep (two-way shear) action. The suggested relationship for two-way shear is

$$V_{c_{CRSI}} = \frac{d}{2w}\left(1 + \frac{d}{c}\right) 4\sqrt{f_c'}\, b_o d \le 32\sqrt{f_c'}\, b_o d \qquad (2)$$

where b_o equals $4 \times c$ for a square column of dimension c. As the critical section is at the perimeter of the column, the CRSI two-way shear strength equation is much more sensitive to the dimensions of the column compared to the ACI approach, where the critical section is at $d/2$ from the column perimeter [b_o equals $4 \times (c + d)$]. The term $(1 + d/c)$ in the CRSI equation is a factor that compensates for this difference.

Strut-and-tie model

The influence of a concentrated load within d from the face of the support of a member subjected to one-way shear is summarized in Fig. 1. The sectional shear force in such a member is very different depending on which side of the concentrated load the "critical section" is located on [see Fig. 1(b)]. The truss model shown in Fig. 1(d) indicates that the concentrated load is transmitted directly to the support by a compression strut. No stirrups are required to resist the "shear" created by the concentrated load [see Fig. 1(f)]. The concentrated load does, however, increase the diagonal compression stresses in the concrete immediately above the support [see Fig. 1(e)], as well as the required tension force in the longitudinal reinforcement at the face of the support [see Fig. 1(g)]. Fig. 2 depicts a simple three-dimensional strut-and-tie model for a four-pile cap. The concentrated column load is transmitted directly to the support by inclined compression struts. Horizontal tension ties (longitudinal reinforcement) are required to prevent the piles from being spread apart.

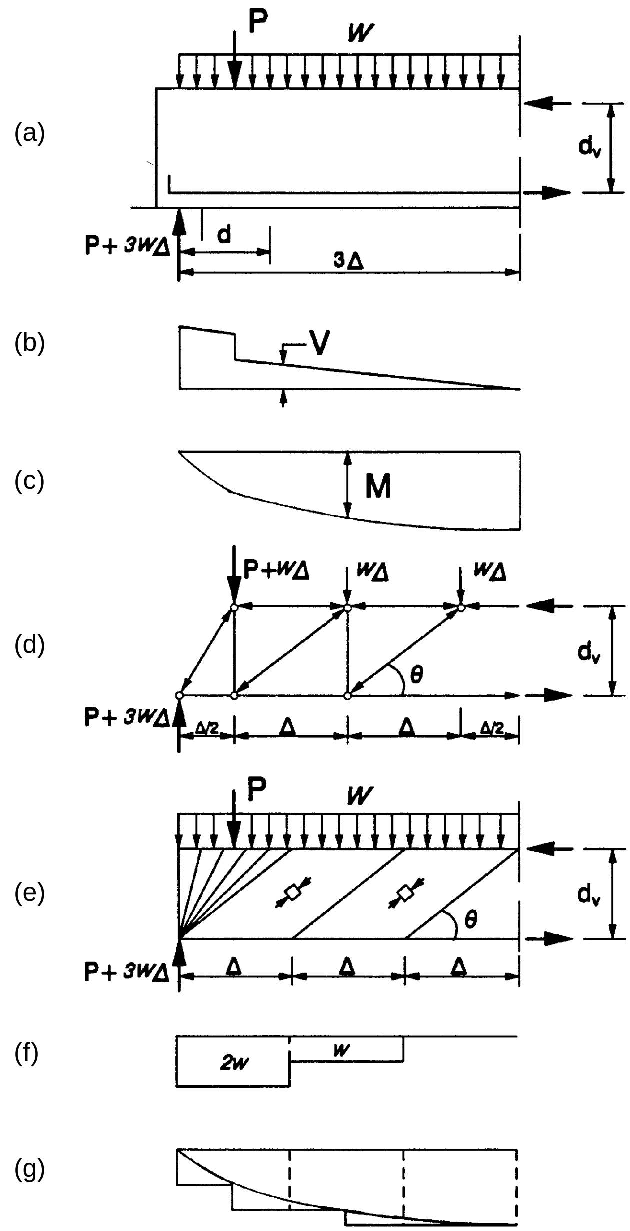


Fig. 1—Truss model for simply supported beam with concentrated load close to support: (a) geometry and loading; (b) sectional shear forces; (c) sectional bending moments; (d) truss model; (e) discontinuous stress field; (f) required stirrup resistance per unit length of beam; (g) required longitudinal reinforcement (adapted from Marti[3])

The "shear design" of a deep pile cap using a strut-and-tie model involves limiting the concrete stresses in compression struts and nodal zones to insure that the tension tie (longitudinal reinforcement) yields prior to any significant diagonal cracking in the plain concrete compression struts. Schlaich et al.[5] suggest that the concrete stresses within an entire disturbed region can be considered safe if the maximum bearing stress in all nodal zones is below a certain limit. Based on an analytical and experimental study of compression struts confined by plain concrete,[9] it is proposed that the maximum bearing stresses in nodal zones of deep pile caps be limited to

$$f_b \le 0.6 f_c' + \alpha\beta 72\sqrt{f_c'} \qquad (3a)$$

$$\alpha = \frac{1}{3}\left(\sqrt{A_2/A_1} - 1\right) \le 1.0 \qquad (3b)$$

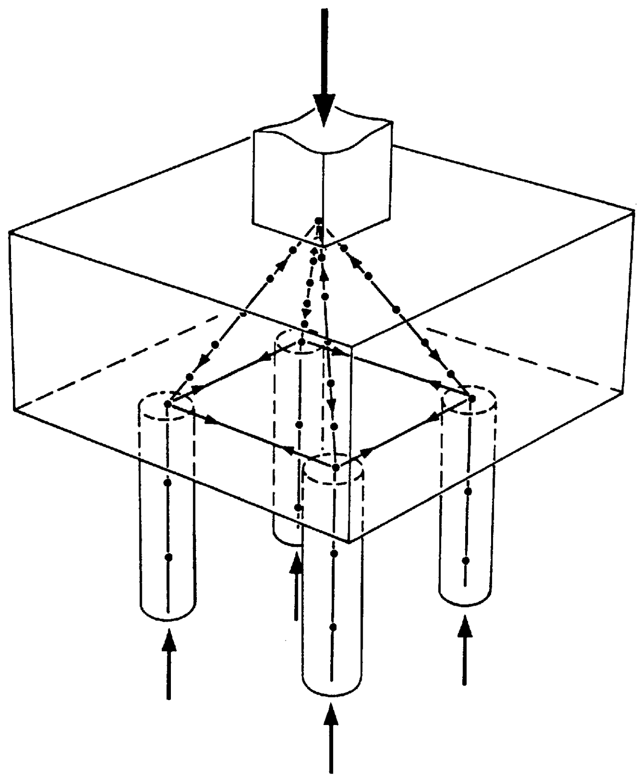


Fig.2—Simple three-dimensional truss model for four-pile cap

$$\beta = \frac{1}{3}\left(\frac{h_s}{b_s} - 1\right) \leq 1.0 \tag{3c}$$

where f_c' and f_b have units of psi. If MPa units are used, the 72 in Eq. (3a) should be replaced by 6. The parameter β accounts for confinement of the compression strut. The ratio A_2/A_1 in Eq. (3b) is identical to that used in the ACI Code to calculate bearing strength. The parameter β accounts for the geometry of the compression strut, where the ratio h_s/b_s is the aspect ratio (height-to-width) of the compression strut. To calculate the maximum bearing stress for the nodal zone below a column, where two or more compression struts meet, the aspect ratio of the compression strut can be approximated as

$$\frac{h_s}{b_s} \approx \frac{2d}{c} \tag{4}$$

where d is the effective depth of the pile cap and c is the dimension of a square column. For a round column, the diameter may be used in place of c. To calculate the maximum bearing stress for a nodal zone above a pile, where only one compression strut is anchored, the aspect ratio of the compression strut can be approximated as

$$\frac{h_s}{b_s} \approx \frac{d}{d_p} \tag{5}$$

where d_p is the diameter of a round pile. Note that the ratio h_s/b_s should not be taken less than 1 (i.e., $\beta \geq 0$).

The lower bearing stress limit of 0.6 f_c' in Eq. (3) is appropriate if there is no confinement ($A_2/A_1 \approx 1$), regardless of the height of the compression strut, as well as when the compression strut is short ($h_s/b_s \approx 1$), regardless of the amount of confinement. The upper limit of Eq. (3) results in similar maximum bearing strengths as the ACI Code.

The proposed strut-and-tie model approach is intended for the design of deep pile caps, not slender pile caps. As it is not always obvious whether a pile cap is slender or deep, and some pile caps may be somewhere in between, a general shear design procedure for pile caps can be accomplished by the following. First, choose the initial pile cap depth using the traditional ACI Code one-way and two-way shear design procedures. In the case of one-way shear, the critical section should be taken at d from the column face, and any pile force within the critical section should be ignored (i.e., the ACI procedure prior to 1983). Second, the nodal zone bearing stresses should be checked using Eq. (3). If necessary, the pile cap depth may be increased (β increased), or the pile cap dimensions may be increased to increase the confinement of the nodal zones (α increased), or else the bearing stresses may need to be reduced by increasing the column or pile dimensions. Thus, the shear strength of slender pile caps will be limited by the traditional sectional shear design procedures, while the shear strength of deep pile caps will be limited by the nodal zone bearing stress limits.

Comparison of design methods

To compare the one-way shear design procedures, Fig. 3 summarizes the relationship between the maximum column load and the width b and depth d of a two-pile cap. When the width of the pile cap is the same as the column width ($b = c$), the pile cap is essentially a deep beam [see Fig. 3(b)]. When the width of the pile cap is increased, larger shear forces can be resisted by the increased concrete area at the critical section, and the maximum bearing stress (and hence, maximum column load) is larger as a result of increased confinement [see Fig. 3(c) and (d)].

Three different ACI Code predictions for one-way shear are given in Fig. 3. The least conservative prediction, entitled "ACI '77," is what designers of pile caps could have used prior to the 1983 edition of the ACI Building Code (any pile within d of the column face is assumed to produce no shear on the critical section); the "ACI '83" procedure is what designers must use since the 1983 edition of the ACI Code (critical section at the column face). This method gives very conservative predictions. The procedure from Section 11.8 for deep flexural members, "ACI [11.8]," gives an intermediate result. The CRSI Handbook method, in which the critical section is also at the face of the column, is much less conservative than "ACI '83" due to an enhanced concrete contribution, but it's more conservative than when the critical section is taken at d from the column face ("ACI '77").

All methods predict that when the pile cap is very deep, the maximum column load is limited by bearing strength (indicated by the horizontal lines in Fig. 3). When the pile cap is twice as wide as the column ($b = 2c$), the ACI Code predicts that confinement is sufficient so that the bearing strength has reached the upper limit of $2 \times 0.85 f_c' = 1.7 f_c'$. Results from numerous bearing strength tests and the procedure proposed

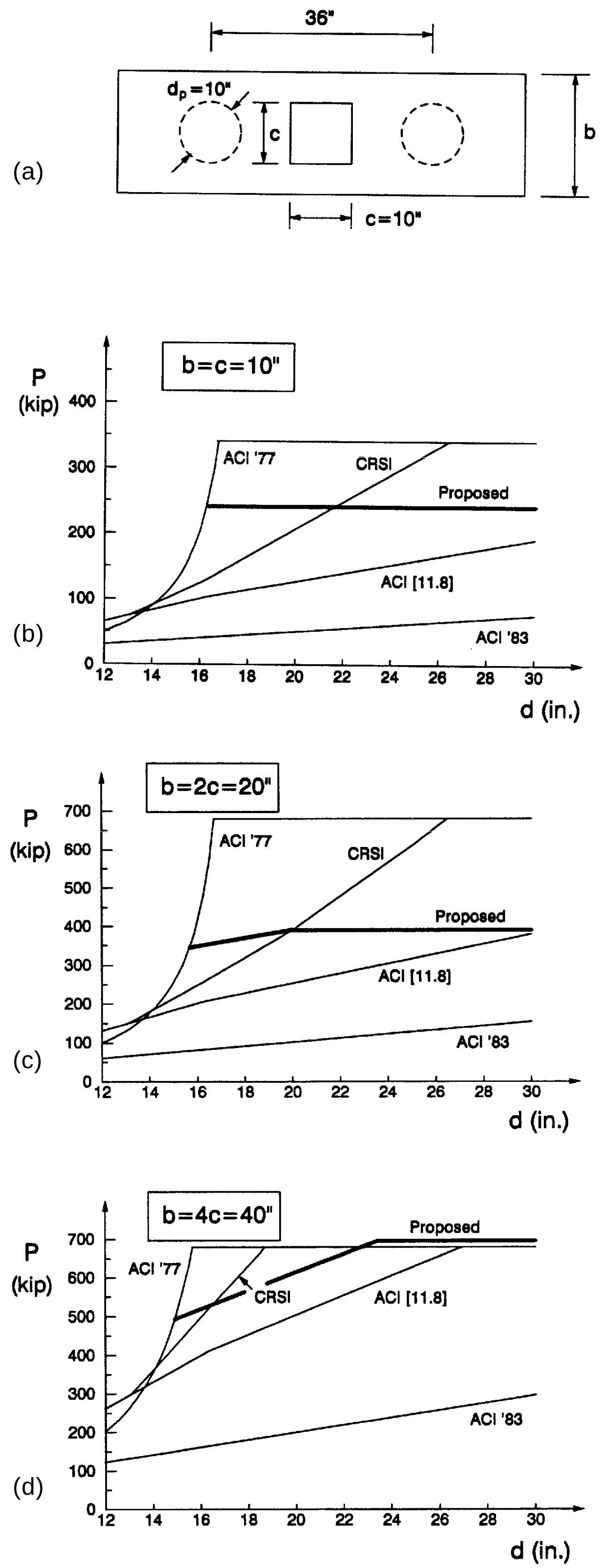


Fig. 3—Comparison of one-way shear design methods for two-pile caps with f_c' = 25 MPa: (a) plan view of pile cap; (b) to (d) influence of pile cap depth on column load for various pile cap widths (1 in. = 25.4 mm; 1 kip = 4.45 kN)

by Hawkins[12] (which is the origin of the ACI Code procedure) indicate that the increase in bearing strength due to confinement is more gradual than suggested by the ACI

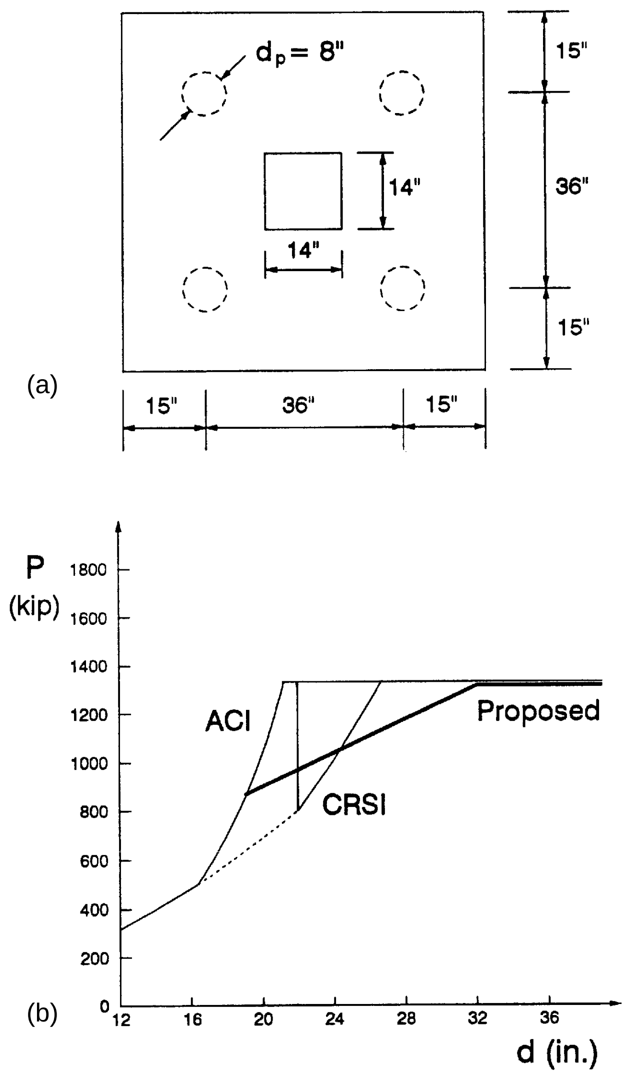


Fig. 4—Comparison of two-way shear design methods for typical four-pile cap with f_c' = 25 MPa: (a) plan view of pile cap; (b) influence of pile cap depth on column load (1 in. = 25.4 mm; 1 kip = 4.45 kN)

Code. That is, when $b = 2c$ the confinement may not be sufficient to support a column bearing stress of 1.7 f_c' (a detailed discussion of this issue was recently presented by the authors[9]).

Fig. 4 compares the influence of pile cap depth on two-way shear strength predictions for a typical four-pile cap. Although the CRSI Handbook expression gives a considerably larger concrete contribution for deep pile caps than the ACI Code, the maximum column load is always smaller than the ACI Code method. This is because in the ACI Code method, the critical section is at $d/2$ from the column face and any pile that intercepts the critical section is assumed to transmit part of the load directly to the column. For example, if a pile is centered on the critical section, only half of the pile reaction must be resisted by the critical section according to the ACI Code method. It is interesting to note that as the CRSI Handbook method suggests that the ACI Code procedures be used until the center of the nearest pile is at $d/2$ from the column face, there is an abrupt reduction in maximum column load at that point (d = 22 in. in Fig. 4). This problem can be corrected by applying the CRSI Handbook procedure when the face of the pile is within $d/2$ of the column face so that none of the pile shear bypasses the critical section; the result is shown by the dashed line in Fig. 4.

The proposed method, which combines the "ACI '77" procedure for pile caps with smaller depths (slender pile caps) with the more conservative bearing stress limit in Eq. (3) gives a very reasonable result. Note that for the particular example shown in Fig. 4, the pile bearing stress is slightly more critical than column bearing stress. That is, according to the proposed method, the confinement around the pile is not sufficient to reach the maximum bearing stress limit.

EXPERIMENTAL RESULTS

The first results from tests on pile caps were reported by Hobbs and Stein[13] who tested numerous small-scale models of two-pile caps. In all cases, the simulated column and piles were the same width as the "pile cap," so the models were really wide deep beams. The models had various amounts of either straight or curved nondeformed reinforcing bars that were anchored by a number of different methods. Shear failure occurred when a diagonal crack formed between the column and a pile.

Deutsch and Walker[14] tested four full-scale two-pile cap specimens. The objective of the tests was to investigate the influence of pile cap depth and the amount of reinforcing steel. Specimens were stronger than anticipated, and two of the specimens did not fail. All pile caps behaved similarly with one main vertical (flexural) crack forming at midspan.

Blévot and Frémy[7] tested two series of pile caps. The first series consisted of 94 models at about half-scale, while the second series consisted of 22 approximately full-scale specimens (eight four-pile caps, eight three-pile caps, and six two-pile caps). The main objective of the tests was to determine the influence of pile cap depth and longitudinal reinforcement layout. The longitudinal reinforcement was either concentrated over the piles, as suggested by a truss model, or distributed in a uniform orthogonal grid, as required by the ACI Code.

Bunching the longitudinal reinforcement resulted in higher capacities (for a given quantity of steel), even though some parts of the specimens had poor crack control. Distributing an equal amount of reinforcement in a uniform grid resulted in the four-pile caps being 20 percent weaker and the three-pile caps being 50 percent weaker. The capacities were not significantly influenced by whether the bunched reinforcement was provided around the perimeter of the pile cap or diagonally across the pile cap; however, the best crack control under service loads occurred when a combination of the two was used.

Clarke[8] tested 15 four-pile caps, all approximately half-scale. The longitudinal reinforcement layout and anchorage were the parameters studied. Similar to Blévot and Frémy, the reinforcement was either bunched over the piles or distributed in a uniform grid. In the study, "nominal anchorage" involved extending the longitudinal reinforcement just beyond the piles, while "full anchorage" meant providing a 90-deg hook and extending the longitudinal reinforcement to the top of the pile cap.

The behavior of all pile caps was similar. Vertical cracks formed near the center of the pile cap sides, extending to near the top of the pile caps. Prior to failure, the pile caps had usually split into four separate pieces hinged below the column base. According to the author, most specimens failed in "shear" after the longitudinal reinforcement yielded. The author also classified the failure modes as either one-way (beam) shear or two-way (punching) shear, depending on the appearance of the failed specimen. Bunching the reinforcement over the piles resulted in a 14 percent increase in capacity compared to spreading the reinforcement uniformly. The so-called "full anchorage" resulted in approximately a 30 percent increase in capacity.

Sabnis and Gogate[15] tested nine very small ($^1/_5$) scale models of four-pile caps to study how the quantity of uniformly distributed longitudinal reinforcement influences the shear capacity of deep pile caps. Similar to Clarke,[8] the longitudinal reinforcement was hooked and extended to the top surface. The tests showed that varying the reinforcement ratio between 0.0014 and 0.012 had little influence on the shear capacities of the models; however, no details were given about how artificial restraint was eliminated at the base of the simulated piles.

Adebar, Kuchma, and Collins[16] tested six full-scale pile caps (five four-pile caps and one six-pile cap). The largest specimen weighed more than 7 ton (6.4 tonne). All pile caps were statically indeterminate (piles in four-pile caps were arranged in a diamond shape), and the actual pile loads were measured throughout the test. Sliding bearings were used under the pseudo-piles to simulate the lateral flexibility of piles. External and internal strain measurements taken during the tests demonstrated that the behavior of pile caps is very different from two-way slabs. Plane sections do not remain plane, and strut action is the predominant mechanism of shear resistance. Deep pile caps deform very little before failure and thus, have virtually no ability to redistribute pile loads.

Strain gages in two of the specimens indicated that the longitudinal reinforcement had definitely yielded prior to failure; however, the failure mode still looked very much like a "shear failure" because the plain concrete in the pile caps had very little ductility. The authors believed that true shear failures (prior to steel yielding) were a result of compression struts splitting longitudinally. Depending on the geometry of the pile cap, the final failure mechanism resembled either a one-way or two-way shear failure. The maximum bearing stress in specimens that failed in shear varied from 1.13 to 1.27 f_c'.

COMPARATIVE STUDY

Table 1 summarizes the properties of 48 pile cap specimens that are used in the comparative study. Specimens not considered include the small wide-beam models tested by Hobbs and Stein, the small-scale specimens (first series) tested by Blévot and Frémy, and the two specimens tested by Deutsch and Walker that did not fail.

Table 2 summarizes the details of the ACI Code and CRSI Handbook predictions. In the case of one-way shear, three different predictions are given from the ACI Building Code: 1) the 1977 edition of the ACI Building Code (critical section at d from the column face); 2) the 1983 ACI Building Code (critical section at the column face); and 3) the special provisions for deep flexural members (Section 11.8 of the

Table 1—Summary of pile cap test results

Specimen	No. of piles	d, mm	Column size, mm	Pile size, mm	f_c', MPa	Reinforcement layout	Failure load, kN
Blévot and Frémy[7]							
2N1	2	495	350 square	350 square	23.1	Bunched	2059
2N1b	2	498	350 square	350 square	43.2	Bunched	3187
2N2	2	703	350 square	350 square	27.3	Bunched	2942
2N2b	2	698	350 square	350 square	44.6	Bunched	5100
2N3	2	894	350 square	350 square	32.1	Bunched	4413
2N3b	2	892	350 square	350 square	46.1	Bunched	5884
3N1	3	447	450 square	350 square	44.7	Bunched	4119
3N1b	3	486	450 square	350 square	44.5	Bunched	4904
3N3	3	702	450 square	350 square	45.4	Bunched	6080
3N3b	3	736	450 square	350 square	40.1	Bunched	6669
4N1	4	674	500 square	350 square	36.5	Bunched and grid	6865
4N1b	4	681	500 square	350 square	40.0	Bunched and grid	6571
4N2	4	660	500 square	350 square	36.4	Bunched	6453
4N2b	4	670	500 square	350 square	33.5	Bunched	7247
4N3	4	925	500 square	350 square	33.5	Bunched and grid	6375
4N3b	4	931	500 square	350 square	48.3	Bunched and grid	8826
4N4	4	920	500 square	350 square	34.7	Bunched	7385
4N4b	4	926	500 square	350 square	41.5	Bunched	8581
Deutsch and Walker[14]							
3	2	533	165 square	254^2	23.8	Bunched	596
4	2	373	165 square	254^2	23.6	Bunched	289
Clarke[8]							
A1	4	400	200 square	200 round	20.9	Grid	1110
A2	4	400	200 square	200 round	27.5	Bunched	1420
A3	4	400	200 square	200 round	31.1	Bunched	1340
A4	4	400	200 square	200 round	20.9	Grid	1230
A5	4	400	200 square	200 round	26.9	Bunched	1400
A6	4	400	200 square	200 round	26.0	Bunched	1230
A7	4	400	200 square	200 round	24.2	Grid	1640
A8	4	400	200 square	200 round	27.5	Bunched	1510
A9	4	400	200 square	200 round	26.8	Grid	1450
A10	4	400	200 square	200 round	18.2	Grid	1520
A11	4	400	200 square	200 round	17.4	Grid	1640
A12	4	400	200 square	200 round	25.3	Grid	1640
B1	4	400	200 square	200 round	26.9	Grid	2080
B3	4	400	200 square	200 round	36.3	Grid	1770
Sabnis and Gogate[15]							
SS1	4	111	76 round	76 round	31.3	Grid	250
SS2	4	112	76 round	76 round	31.3	Grid	245
SS3	4	111	76 round	76 round	31.3	Grid	248
SS4	4	112	76 round	76 round	31.3	Grid	226
SS5	4	109	76 round	76 round	41.0	Grid	264
SS6	4	109	76 round	76 round	41.0	Grid	280
SG2	4	117	76 round	76 round	17.9	Grid	173
SG3	4	117	76 round	76 round	17.9	Grid	177
Adebar, Kuchma, and Collins[16]							
A	4	445	300 square	200 round	24.8	Grid	1781
B	4	397	300 square	200 round	24.8	Bunched	2189
C	6	395	300 square	200 round	27.1	Bunched	2892
D	4	390	300 square	200 round	30.3	Bunched	3222
E	4	410	300 square	200 round	41.1	Bunched and grid	4709
F	4	390	300 square	200 round	30.3	Bunched	3026

Table 2—Summary of ACI Building Code and CRSI Handbook predictions

		Bearing		One-way shear				Two-way shear		
				ACI				Column		
Specimen	Flexure	Column	Pile	1977	1983	(11.8)	CRSI	ACI	CRSI	Pile
2N1	2197	2749	5498	1049*	314	951	775	‡	‡	‡
2N1b	3756	5141	10,282	1442*	432	1295	902	‡	‡	‡
2N2	3432	3249	6498	†	490	1461	2432	‡	‡	‡
2N2b	5551	5308	10,616	†	618	1844	2628	‡	‡	‡
2N3	5413	3820	7640	†	677	2020	3364	‡	‡	‡
2N3b	7257	5487	10,974	†	804	3364	4021	‡	‡	‡
3N1	3825	15,388	23,877	2128*	1589*	4492	2020	3717*	6551	†
3N1b	5286	15,319	23,770	2697*	1716*	4737	2638	4394*	8061	†
3N3	6129	15,629	24,251	†	2511*	7493	9317	†	20,918	†
3N3b	7983	13,804	21,420	†	2471*	7385	9876	†	22,252	†
4N1	7924	15,513	25,996	†	2824	7257	11,866	11,852*	§	†
4N1b	8159	17,000	28,489	†	2766	7689	11,965	12,749*	§	†
4N2	7542	15,470	25,925	†	2373	7139	11,307	11,003*	§	†
4N2b	8552	14,238	23,859	†	2314	6953	10,670	11,102*	§	†
4N3	8277	14,238	23,859	†	3609	9650	16,083	59,607*	13,220	†
4N3b	10,807	20,528	34,400	†	4080	11,239	19,320	71,621*	16,309	†
4N4	9866	14,748	24,714	†	3236	9709	16,182	54,998*	13,426	†
4N4b	10,866	17,638	29,557	†	3560	10,435	17,819	63,746*	14,937	†
No. 3	512	1102	3915	†	343	925	560	‡	‡	‡
No. 4	271	1092	3883	†	231	503	§	‡	‡	‡
A1	1258	1421	3907	†	604	1646	2718	2916*	1458	1996
A2	1266	1870	5140	†	684	1847	3078	3344*	1672	2288
A3	1256	2115	5813	†	722	1934	3250	3558*	1778	2434
A4	1258	1421	3907	†	604	1646	2718	2916*	1458	1996
A5	1265	1829	5028	†	678	1830	3052	3308*	1654	2263
A6	1252	1768	4860	†	664	1791	2988	3252*	1626	2225
A7	1262	1646	4524	†	644	1750	2898	3138*	1569	2148
A8	1266	1870	5140	†	684	1847	3078	3345*	1672	2288
A9	1264	1822	5010	†	676	1828	3042	3302*	1651	2260
A10	1252	1238	3402	†	566	1554	2548	2722*	1360	1860
A11	1252	1183	3253	†	556	1526	2502	2660*	1330	1820
A12	1262	1720	4729	†	658	1784	2962	3208*	1604	2196
B1	2022	1829	5028	†	578	2066*	2584	†	3308	†
B3	1528	2468	6785	†	636	2338*	3002	†	3843	†
SS1	133	241	806	†	69	186	256	122	§	228
SS2	116	241	806	†	68	178	252	122	§	228
SS3	194	241	806	†	69	181	251	121	§	226
SS4	158	241	806	†	71	192	262	122	§	228
SS5	317	316	1056	†	84	229	287	134	§	251
SS6	455	316	1056	†	89	229	305	134	§	251
SG2	302	138	461	†	65	164	254	101	§	185
SG3	628	138	461	†	85	164	329	101	§	185
A	2256	3794	5298	3246	2397	6056	6349	2309*	§	6247
B	2790	3794	5298	3411	2085	5308	4269	1839	§	2762
C	4009	4146	8684	6300	1820	4938	3740	1899	§	2990
D	5646	4636	6473	3773	2431	6348	4724	1968	§	3106
E	7428	6288	8780	4475	3076	8141	7058	2475	§	3970
F	5324	3083	6473	1604	573	1739	1619	‡	‡	‡

*Increased capacity since piles partially within critical section.
†Infinite capacity since piles totally within critical section.
‡Procedure not applicable.
§CRSI prediction not applicable (use ACI).

ACI Code). Table 3 presents the ratio of measured pile cap capacity to predicted capacity for the three ACI Code predictions, as well as the CRSI Handbook prediction. The predicted failure mode and reported failure mode are also given. It is interesting to note that many pile caps predicted to fail in flexure were reported to have failed in shear. As previously mentioned, the likely reason for this is that pile caps are large blocks of plain concrete that do not have the ductility to un-

Table 3—Comparison of ACI Code and CRSI Handbook predictions: ratio of measured capacity to predicted capacity and failure mode*

Name	ACI '77	ACI '83	ACI (11.8)	CRSI	Reported failure mode
2N1	1.96 s_1	6.56 s_1	2.17 s_1	2.66 s_1	s
2N1b	2.21 s_1	7.38 s_1	2.46 s_1	3.53 s_1	s
2N2	0.91 b_c	6.00 s_1	2.01 s_1	1.21 s_1	s
2N2b	0.96 b_c	8.25 s_1	2.77 s_1	1.94 s_1	s
2N3	1.16 b_c	6.52 s_1	2.18 s_1	1.31 s_1	s
2N3b	1.07 b_c	7.32 s_1	1.75 s_1	1.46 s_1	s
3N1	1.94 s_1	2.59 s_1	1.11 s_2	2.04 s_1	s
3N1b	1.82 s_1	2.86 s_1	1.04 s_1	1.86 s_1	s
3N3	0.99 f	2.42 s_1	0.99 f	0.99 f	s
3N3b	0.84 f	2.70 s_1	0.90 s_1	0.84 f	s
4N1	0.87 f	2.43 s_1	0.95 s_1	0.87 f	s
4N1b	0.81 f	2.38 s_1	0.85 s_1	0.81 f	s
4N2	0.86 f	2.72 s_1	0.90 s_1	0.86 f	s
4N2b	0.85 f	3.13 s_1	1.04 s_1	0.85 f	s
4N3	0.77 f	1.77 s_1	0.77 f	0.77 f	s
4N3b	0.82 f	2.16 s_1	0.82 f	0.82 f	s
4N4	0.75 f	2.28 s_1	0.76 s_1	0.75 f	s
4N4b	0.79 f	2.41 s_1	0.82 s_1	0.79 f	s
No. 3	1.16 f	1.74 s_1	1.16 f	1.16 f	s
No. 4	1.07 f	1.25 s_1	1.07 f	1.07 f	s
A1	0.88 f	1.84 s_1	0.88 f	0.88 f	s
A2	1.12 f	2.08 s_1	1.12 f	1.12 f	s
A3	1.07 f	1.86 s_1	1.07 f	1.07 f	s
A4	0.98 f	2.04 s_1	0.98 f	0.98 f	s
A5	1.11 f	2.06 s_1	1.11 f	1.11 f	s
A6	0.98 f	1.85 s_1	0.98 f	0.98 f	s
A7	1.30 f	2.55 s_1	1.30 f	1.30 f	s
A8	1.19 f	2.21 s_1	1.19 f	1.19 f	s
A9	1.15 f	2.14 s_1	1.15 f	1.15 f	s
A10	1.23 b_c	2.69 s_1	1.23 b_c	1.23 b_c	f
A11	1.39 b_c	2.95 s_1	1.39 b_c	1.39 b_c	f
A12	1.30 f	2.49 s_1	1.30 f	1.07 f	f
B1	1.14 f	3.60 s_1	1.14 b_c	1.14 f	s
B3	1.16 f	2.78 s_1	1.16 f	1.16 f	f
SS1	2.05 s_2	3.62 s_1	2.05 s_2	2.05 s	s
SS2	2.11 f	3.60 s_1	2.11 f	2.11 f	s
SS3	2.05 s_2	3.59 s_1	2.05 f	2.05 f	s
SS4	1.85 s_2	3.18 s_1	1.85 s_1	1.85 s_1	s
SS5	1.97 s_2	3.14 s_1	1.97 s_2	1.97 s_2	s
SS6	2.09 s_2	3.15 s_1	2.09 s_2	2.09 s_2	s
SG2	1.71 s_2	2.66 s_1	1.71 s_2	1.71 s_2	s
SG3	1.75 s_2	2.08 s_1	1.75 s_2	1.75 s_2	s
A	0.79 f	0.79 f	0.79 f	0.79 f	f
B	1.19 s_2	1.19 s_2	1.19 s_2	1.19 s_2	s
C	1.52 s_2	1.59 s_1	1.52 s_2	1.52 s_2	s
D	1.64 s_2	1.64 s_2	1.64 s_2	1.64 s_2	s
E	1.90 s_2	1.90 s_2	1.90 s_2	1.90 s_2	s
F	1.89 s_1	5.28 s_1	1.74 s_1	1.87 s_1	s

Note: f = flexure; b_c = column bearing; s_1 = one-way shear; s_2 = two-way shear; s = shear.

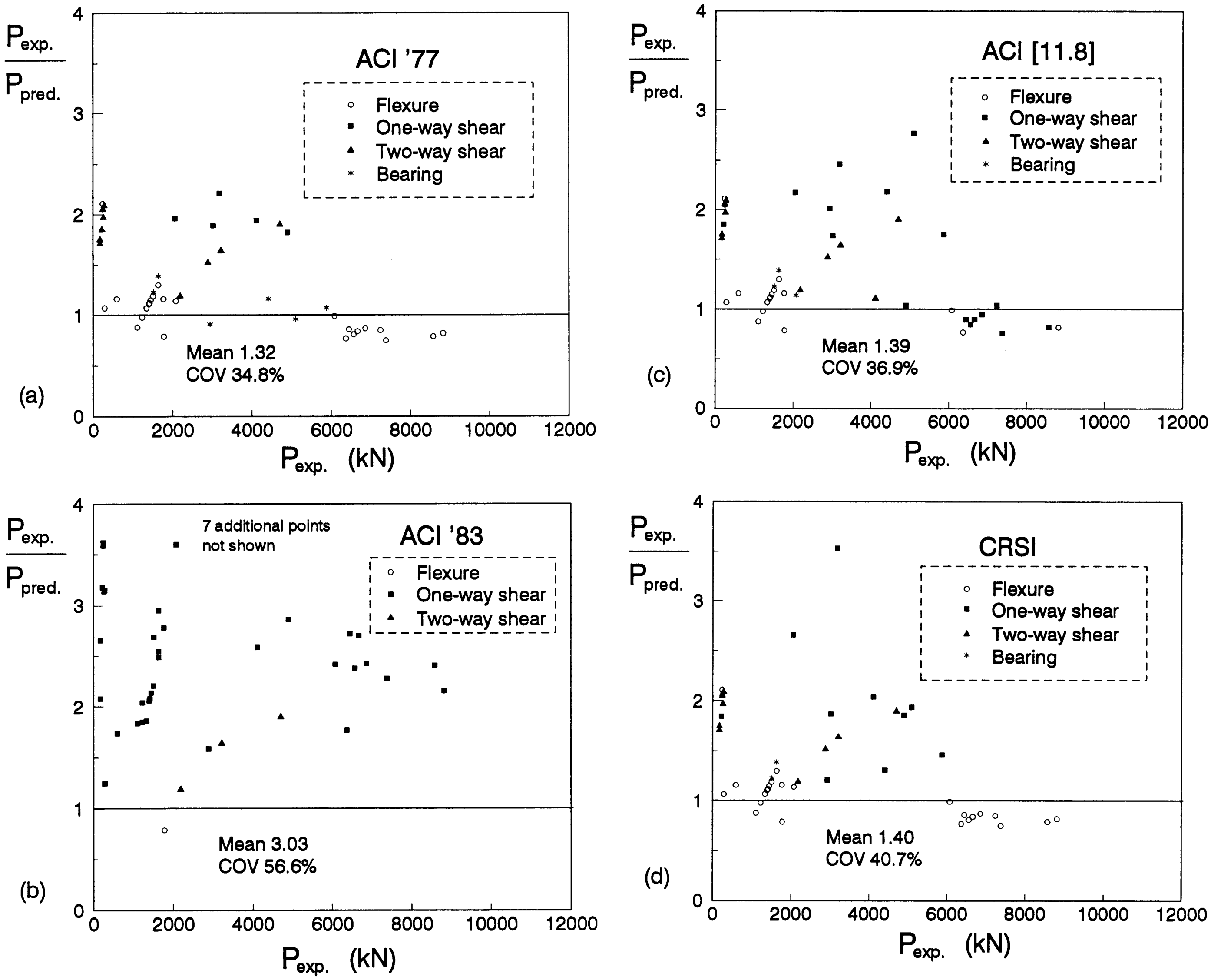
Table 4—Comparison of proposed strut-and-tie model predicitons with experimental results

Name	Predicted		Experimental	Experimental / Predicted
	Flexure	Shear		
2N1	2127	1049a	2059	1.96 s
2N1b	3567	1442a	3187	2.21 s
2N2	3107	2156	2942	1.36 s
2N2b	5047	3470	5100	1.47 s
2N3	4831	2560	4413	1.72 s
2N3b	6439	3623	5884	1.62 s
3N1	3254	2128a	4119	1.94 s
3N1b	4528	2697a	4904	1.82 s
3N3	5067	7493	6080	1.20 f
3N3b	6762	6885	6669	0.99 f
4N1	6037	9050	6865	1.14 f
4N1b	6174	9826	6571	1.06 f
4N2	5929	8877	6453	1.09 f
4N2b	6507	8377	7247	1.11 f
4N3	6203	10,600	6375	1.03 f
4N3b	7007	14,050	8826	1.26 f
4N4	7409	10,900	7385	1.00 f
4N4b	8144	12,450	8581	1.05 f
No. 3	480	732	596	1.24 f
No. 4	253	730	289	1.14 f
A1	1029	1424	1110	1.08 f
A2	1030	1717	1420	1.38 f
A3	1020	1871	1340	1.31 f
A4	1029	1424	1230	1.20 f
A5	1030	1691	1400	1.36 f
A6	1020	1652	1230	1.21 f
A7	1029	1573	1640	1.59 f
A8	1030	1717	1510	1.47 f
A9	1029	1688	1450	1.41 f
A10	1029	1296	1520	1.48 f
A11	1029	1260	1640	1.59 f
A12	1029	1620	1640	1.59 f
B1	1376	1596	2080	1.51 f
B3	1031	1977	1770	1.72 f
SS1	96	122a	250	2.60 f
SS2	85	122a	245	2.88 f
SS3	144	121a	248	2.05 s
SS4	116	122a	226	1.95 f
SS5	237	134a	264	1.97 s
SS6	346	134a	280	2.09 s
SG2	231	101a	173	1.71 s
SG3	543	101a	177	1.75 s
A	1445	1924	1781	1.23 f
B	1662	1696	2189	1.32 f
C	1502	1639	2892	1.93 f
D	3454	1968a	3222	1.64 s
E	5085	2731	4709	1.72 s
F	3472	1303	3026	2.32 s

Note: a = ACI '77 prediction critical; s = shear critical; f = flexure critical.

dergo significant flexural deformations without triggering a shear failure.

Table 4 summarizes the predictions[17] from the proposed strut-and-tie model and compares the predictions with the ex-



perimental results. The "shear" capacity is the maximum column load limited by the nodal zone bearing stresses given by Eq. (3), while the "flexural" capacity is the maximum column load limited by yielding of the longitudinal reinforcement. The flexural capacity depends strongly on the inclination of the compression strut that is defined by the location of the nodal zones. The lower nodal zones were located at the center of the piles at the level of the longitudinal reinforcement, while the upper nodal zones were assumed to be at the top surface of the pile cap at the column quarter points.

Fig. 5 compares the predictions from the various methods. It is obvious from Fig. 5(b) that the one-way shear design provisions of the 1983 and subsequent editions of the ACI Building Code are excessively conservative for pile caps. Fig. 5(a) and 5(d) also demonstrate that the traditional flexural strength predictions are unconservative for pile caps. These flexural strength procedures are meant for lightly reinforced beams that are able to undergo extensive flexural deformations (increased curvatures) after the reinforcement yields. As the curvature increases, the flexural compression stresses concentrate near the compression face of the member. As mentioned previously, pile caps are too brittle to undergo such deformations; therefore, assuming that the flexural

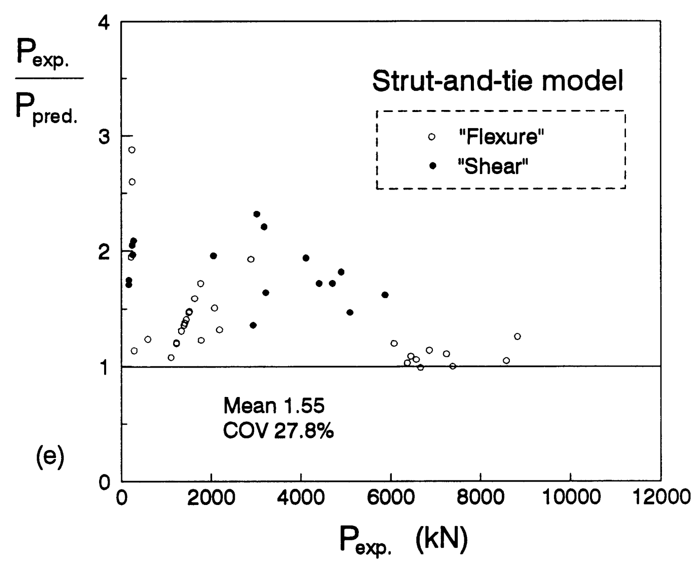


Fig. 5—Ratio of experimentally measured-to-predicted pile cap capacities from: (a) 1977 ACI Building Code (critical section for one-way shear at d from column face); (b) 1983 ACI Building Code (critical section for one-way shear at column face); (c) ACI Building Code special provisions for deep flexural members; (d) CRSI Handbook; (e) proposed strut-and-tie model

compression is concentrated near the compression face is inappropriate. Assuming the flexural compression is uniform across the entire pile cap, which strain measurements have shown to be incorrect,[16] leads to a further overprediction of the flexural capacity.

While the proposed strut-and-tie method gives the least amount of scatter between experimental results and predictions, the amount of scatter is nonetheless still relatively high (COV = 28 percent). This can be explained by the fact that the shear failure of pile caps involves a tension failure of the concrete. It is the author's opinion that a further refinement of the design procedure to reduce this scatter is not warranted. The most important issue is that the proposed design method is simple, rational, and conservative, and unlike the other design methods, it does not overpredict any of the pile cap test results.

SUMMARY AND CONCLUSIONS

Recent editions of the ACI Building Code require that the critical section for one-way shear be taken at the support face if a concentrated load exists within d from the support. While this is appropriate for heavily reinforced deep beams (Fig. 1), where a shear failure may occur due to diagonal crushing of concrete, it is excessively conservative for pile caps [Fig. 5(b)], which do not fail as a result of diagonal compression. The more appropriate one-way shear design procedure for pile caps in the 1977 and earlier editions of the ACI Building Code results in two-way shear and flexure being more critical for most pile caps (except for two-pile caps) [Fig. 5(a)].

The ACI Building Code procedure for two-way shear involves a critical section at $d/2$ from the face of the column, and any pile reaction within $d/2$ from the column face does not produce shear on the critical section. This results in an "infinite" two-way shear capacity for some deep pile caps (Table 2). The CRSI Handbook suggests an alternate two-way shear design procedure for deep pile caps, where the critical section is at the column face. Since the critical section must resist much larger shear forces, the concrete contribution is greatly enhanced to account for deep two-way action. While the sectional shear resistance is larger according to the CRSI Handbook method, the maximum column load is usually smaller than the ACI Code method, where a significant portion of the column load does not produce shear on the critical section.

The CRSI Handbook suggests an upper limit of $32\sqrt{f_c'}$ for the shear stress on two-way critical sections in very deep members and others[18] have suggested reducing this limit to $24\sqrt{f_c'}$. Neither suggestion is based on any experimental results; however, an upper limit is actually not needed since the maximum load that can be applied to very deep pile caps is always limited by bearing stress at either the base of the column or the top of the piles (see Fig. 3).

In this paper a simple rational design method for deep pile caps is proposed in which the maximum bearing stress is considered a better indicator of shear strength than the "shear stress" on any prescribed critical section. In deep pile caps the shear stress is concentrated in zones (compression struts) between the column and piles, and is not uniform over the height, which makes it difficult to calculate a meaningful shear stress. The procedure suggested herein is based on the premise proposed by Schlaich et al.[5] that an entire D-region of a concrete structure can be considered safe if the maximum bearing stress is maintained below a certain limit.

Based on a study of idealized compression struts confined by plain concrete,[9] Eq. (3) is proposed for the maximum bearing stress in pile caps. The maximum bearing stress is a function of confinement (similar to the ACI Code), as well as the aspect ratio (height-to-width) of the compression struts that transmit shear between the column and piles. The influence of confinement is much more gradual in the proposed relationship than in the ACI Code procedure (i.e., more confinement is needed before reaching the maximum bearing stress).

A general shear design procedure for all pile caps (deep or slender) can be accomplished by combining the ACI Code shear design procedure with the maximum bearing stress limit of Eq. (3); the more critical one controls. As the bearing stress limit will always control the "shear strength" of very deep pile caps, the shear force from any pile within the critical section (d or $d/2$) can be ignored with confidence.

Comparisons with experimental results indicate that the traditional flexural design procedures for beams and two-way slabs are unconservative for deep pile caps [Fig. 5(a)]. The flexural compressive stresses within pile caps are concentrated near the column (not spread uniformly across the section), and pile caps are large blocks of plain concrete that cannot undergo significant flexural deformations without triggering brittle shear failure. A more appropriate flexural design procedure for deep pile caps can be achieved by using strut-and-tie models. Reasonably conservative designs are obtained [Fig. 5(e)] when the upper nodal zones are located on the top surface of the pile cap at $c/4$ from the column center. Previous experimental results have demonstrated that concentrating the longitudinal reinforcement over the piles, as suggested by strut-and-tie models, results in considerably higher flexural capacities compared to when the longitudinal reinforcement is distributed in a uniform grid; however, some of the longitudinal reinforcement should be uniformly distributed to help control cracking.

The method proposed in this paper for the design of deep pile caps has been implemented in the 1995 CPCA Concrete Design Handbook.[19] The pile cap design tables were developed using the method proposed herein, and a number of examples are provided to show how to apply the method in manual calculations.

ACKNOWLEDGMENT

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