Analysis of reinforced concrete pile caps by the finite element method

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This paper presents the application of the Finite Element Method to analyze reinforced concrete pile caps. The analysis was used to study the behavior of 20 pile caps subjected to column loads. The effect of pile cap thickness and piles arrangement on the behavior of such caps are examined. The structural behavior of the considered pile caps was investigated. Such structural behaviour included: flexural and shear stress distributions in the elastic range of loading, load-deflection relationships over the complete range of loading, the initiation and propagation of cracks, and ultimate loads. Also, the results of the elastic analysis were compared with code limitations and conventional methods of analysis. The results revealed that linear elastic analysis may be used for the analysis of pile caps since for most of the pile caps studied no cracks were formed up to the average design working load of the pile group frequently used in practice. Also, it was found that the assumption of equal pile load distribution for five and six-pile caps is appropriate within the elastic range of loading. However, beyond such range of loading this assumption becomes invalid.

تناول هذا البحث إستخدام طريقة العناصر المحددة لتحليل وسائد الخوازيق الخرسانية المسلحة المعرضة لأحمال الأعمدة. فى هذا البحث تم دراسة تأثير سمك الوسادة الخرسانية المسلحة وكذا ترتيب الخوازيق على السلوك الإنشائى لعدد عشرين وسادة وقد شملت النتائج توزيع إجهادات الإنحناء والقص، ترخيم الوسائد تحت تأثير أحمال الأعمدة، ظهور وتطور الشروخ فى الخرسانة وأخيراً الأحمال القصوى. وقد تمت مقارنة النتائج مع الحدود المذكورة فى المواصفات المصرية لتصميم وتنفيذ المنشآت الخرسانية وكذلك مع الطريقة التقليدية المتبعة لتحليل وسائد الخوازيق. وقد أوضحت النتائج أنه يمكن إستخدام طرق التحليل المرنة (الخطية) لهذه الوسائد حيث أنه فى معظم الحالات التى تمت دراستها كان حمل التشريح أكبر من حمل التشغيل لمجموعة الخوازيق. كذلك فإن إفتراض توزيع أحمال الأعمدة توزيعاً منتظماً على مجموعة الخوازيق للوسائد يكون إفتراضاً صحيحاً حتى بداية ظهور الشروخ فى الخرسانة حيث يحدث إعادة لتوزيع أحمال الخوازيق وتكون أحمال الخوازيق تحت العمود أو القريبة منه أكبر من أحمال الخوازيق البعيدة عن الأعمدة.

Keywords: Finite Element, Shear, Pile cap, Reinforced concrete.

1. Introduction

A pile cap is provided to spread vertical and horizontal loads (if any) of a column to a group of piles. The assumption that each pile in a group carries equal load may be nearly correct when the pile group is symmetric and the cap has ample rigidity. Thick reinforced concrete (RC) pile caps may be defined as caps whose thickness; t, is equal to or greater than the distance from centerline of pile to the face of supported column; a, [1]. The ratio of the shear span, a, to the depth of the flexural reinforcement, d, (a/d) for thick pile caps is usually less than unity.

Various methods are used for the analysis of RC pile caps. The most common method is known as the rigid beam method. In this method it is assumed that the cap acts as a rigid beam spanning between the piles and the column load is dispersed through the cap along 1: 1 lines to the mid-depth of the cap. The truss analogy, or strut-and-tie model, has also been used as a method of analyzing the load transfer from the column to the piles [2,3,4]. The difference between the two methods is that the truss analogy favors bands of steel reinforcement while the rigid beam method leads to a uniform grid reinforcement. The Finite Element Method (FEM) has also been used to predict the behavior of thick pile caps [3,5]. Adebar et al. [3] used a linear elastic FE analysis to present shear stress distribution in pile caps, whereas the crack patterns at ultimate loads obtained using the FE analysis were displayed for four-pile caps in a paper by Sabins and Gogate [5]. Jimenez et al. [6] predicted the ultimate load capacity of thick pile caps P_u, based on a non-dimensional analysis of data obtained from

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tests. However, in the suggested formula [6], the effect of cap thickness on the ultimate load capacity of caps was neglected.

Design of RC pile caps according to building codes, such as the Egyptian Code ECP-98 [7] and the American Concrete Institute ACI 318-99 code provisions [8], is based on the procedure for slender footings that can be divided into three separate steps as follows:

1) Shear design, which involves calculating the minimum pile cap depth so that the concrete contribution to shear resistance, V_c is greater than the ultimate shear V_u computed at the code-defined critical section in shear. In this regard, both one-way (beam or flexure) shear and two-way (or punching) shear are to be considered. Both ECP-98 and ACI 318-99 code do not provide any specific recommendations for the shear design of "thick" pile caps. However, in the case of slabs and footings, ECP-98 allows the increase of the concrete contribution in shear; V_c, by 30 % than that for beams.
2) Flexural design, in which the usual assumptions of RC beams are used to calculate the required amount of longitudinal reinforcement at critical section for flexure. This critical section as defined by the codes is to be at the column face.
3) A check of the bearing stress at the base of the column and at the top of the piles.

The main objective of this paper was to investigate the behavior of RC pile caps of different dimensions and thickness by using a nonlinear FE analysis. The results from the analysis were compared to those obtained from the conventional rigid beam method and the provisions made by both the ECP-98 and ACI 318-99 design codes.

2. Analytical study

2.1 The finite element analysis

A nonlinear FE analysis was used in the present study. The element is a specialization of the hexahedral solid element developed by Ahmad et al [9] and is applied for the analysis of both thick and thin plates [10]. The element consists of eight nodes (corner and mid-sides) with three degrees of freedom at each node, a vertical translation; w, a rotation about x-axis; θ_x, and a rotation about y-axis, θ_y. The element was divided across its thickness into eight layers with steel reinforcement smeared into the concrete layer. Perfect bond was assumed between the layers and between steel and concrete. Five values of non-zero stresses (normal stresses σ_x, σ_y, in-plane shear stress τ_{xy}, and transverse shear stresses τ_{yz}, τ_{zx}) were considered. The reinforcing steel is assumed to be elastic-plastic with stiffness only in the bar direction. Concrete in compression is treated as an isotropic linearly elastic-plastic strain hardening material. Concrete in tension is modeled as a linear elastic material without strain softening or tension stiffening effects. Smeared cracking approach was adopted in the present work. Details about material modeling are given in Reference [11]. Vertical springs were added to represent pile head displacement at the nodes where piles are located. The spring stiffness, K was calculated as the elastic deformation of the pile, i.e. K= EA/l, where E = modulus of elasticity of concrete = 20 kN/mm^2, A = cross sectional area of the pile, and l is the pile length which was assumed as 10.0 m. One-eighth of the calculated value of K was provided at each node of the element representing the pile.

2.2 Description of pile caps studied

Three types of pile caps were analyzed in this study, namely: square caps supported on four piles (four-pile cap), square caps supported on five piles with one pile being placed directly under the column (five-pile cap), and rectangular caps supported on six piles (six-pile cap). Different thicknesses were considered for each type of pile caps studied. The effective depth, d, of the cap was varied from 0.93 L (thick caps) to 0.39 L (relatively flexible caps), where L is the distance between the centerlines of piles. The values of cap thickness chosen in the present study are those commonly used in practice. Table 1 and Fig. 1 show the dimensions and geometry for all the pile caps analyzed.

Circular piles of 400 mm diameter were used in the study. However, in the finite element analysis the piles were modeled in the shape of squares having dimensions 350 x 350 mm, giving approximately the same area as the actual circular piles. The column load was applied through a rectangular area having dimensions presented in Fig. 1.

As shown in Fig. 2, each model was divided into 10 elements in both directions with 8 layers across the thickness of each element. Due to symmetry in both geometry and loading, only one quarter of the cap was analyzed.

The material properties used in the analysis were as follows:

concrete cube strength f_{cu} = 25 N/mm^2, concrete tensile strength f_t = 3 N/mm^2 , modulus of elasticity of concrete = 22 kN/mm^2, steel yield stress = 360 N/mm^2 , modulus of elasticity of steel = 200 kN/mm^2 .

3. Presentation and discussion of FE results

A large amount of data was obtained from the finite element analysis. The results included deflections, strains, stresses, and cracking sequence at each load increment for all the cases studied. Only representative results will be presented in this paper.

3.1 Results from the elastic analysis

In this section, the linear (elastic) results obtained from the analysis, considering linear properties for both concrete and steel are discussed. Table 2 presents the bending moments and shear stresses as obtained from both the FE analysis and the rigid beam method. The values given in the table are at a column load P of 2400 kN for 4-pile caps, 3000 kN for 5-pile caps, and 3600 kN for 6-pile caps. The results indicate the following:

i- Designers of pile caps usually assume an equal distribution of load among piles according to the assumptions of the rigid beam method. Values of deflection obtained from the FE analysis indicated that for the 5-pile caps, the pile located directly under the column carried a load of 1.06 to 1.14 compared to that carried by each of the other four piles for L/d = 1.33 (t = 900 mm) to L/d = 2.29 (t = 550 mm), respectively. For the 6-pile caps, the load carried by each of the two piles located closest to the column was only 1.03 to 1.09 of that carried by each of the other piles for L/d = 1.07 (t = 1100 mm) to L/d = 2.08 (t = 600 mm), respectively.

ii- Another important assumption in the rigid beam method is that the full width of the cap uniformly resists the applied bending moments. Fig. 3 displays the variation of flexural stresses, $_x$, for the case of 4-pile caps at a load level close to the working load of the pile group. The stresses are presented at two different sections; near the centerline of the cap (section 1-1, Fig. 2) and near the pile edge (section 2-2, Fig. 2). The variation of these flexural stresses along cap width for the cases of 4 and 5 pile caps is shown in Fig. 4. Figs. 3 and 4 indicate that, along the cap width, the flexural stresses (and hence bending moments) had high values in the central portion of the cap and decreased away from the loaded area. The variation in the stress values was higher for caps with low rigidity. Across the critical section (face of column), the bending moment at about 0.3 L from the free end (i.e. at the edge of the pile) was only 0.3 that at the centerline of the cap (i.e. at 0.5 L). Figure 3 also indicates that away from the central portion of the cap, negative moments (tension top) were obtained. These moments were remarkable for caps with low rigidity and the maximum value of negative flexural stresses occurred at the pile internal corner. Similar behavior was obtained for the case of 5-pile caps. It should be noted that the rigid beam method does not predict such behavior, since in this method it is assumed that the cap is perfectly rigid. As given in Table 2 the average values of bending moments along cap width at the critical

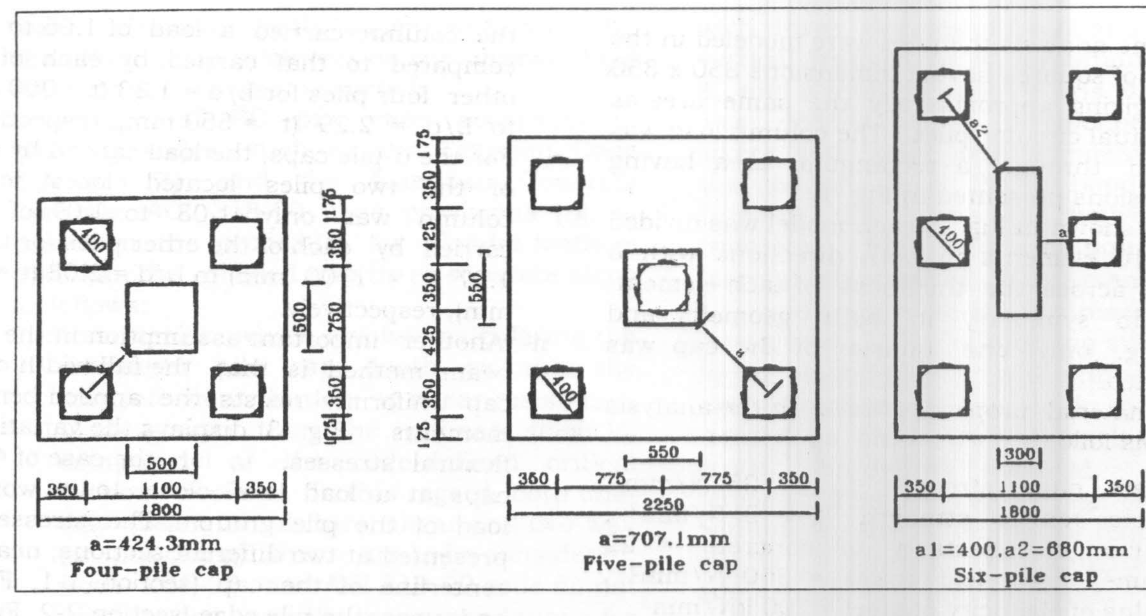

Fig. 1 Geometry of R.C. pile caps.

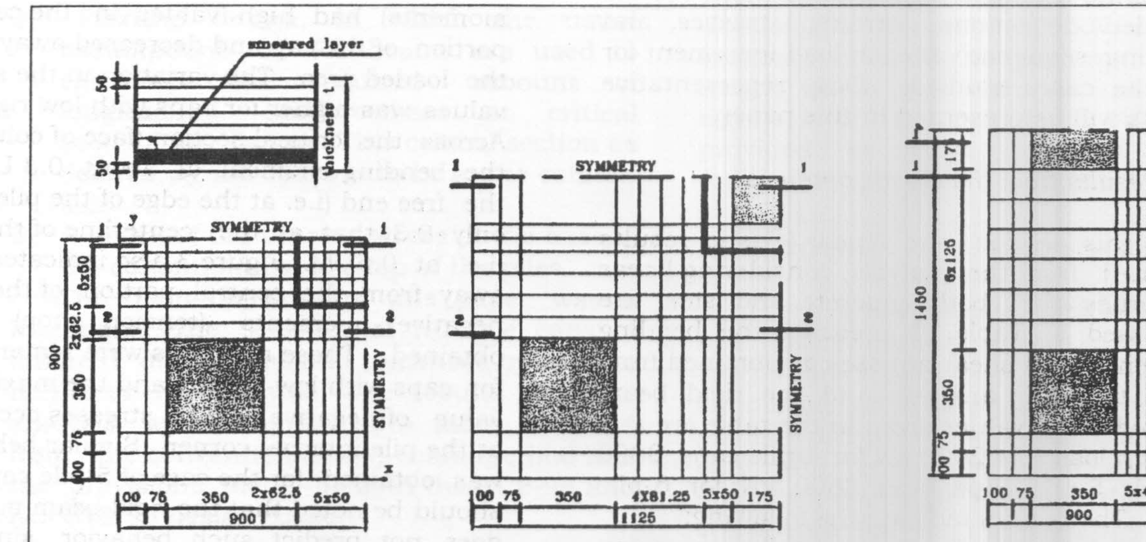

Fig. 2 Finite element mesh used in the analysis.

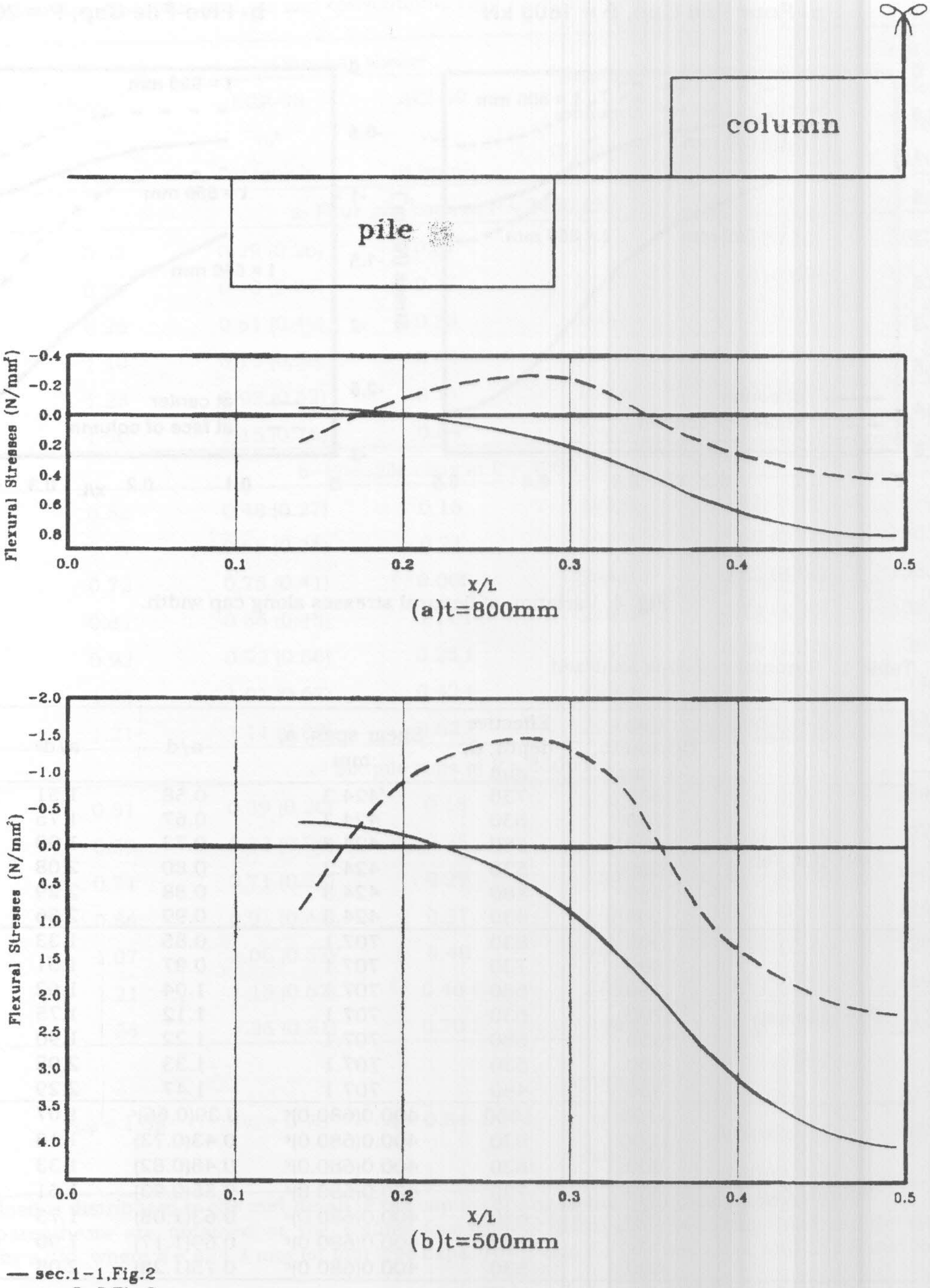

Fig. 3. Variation of flexural stresses, σ_χ, for 4-pile cap, p=1600 kN.

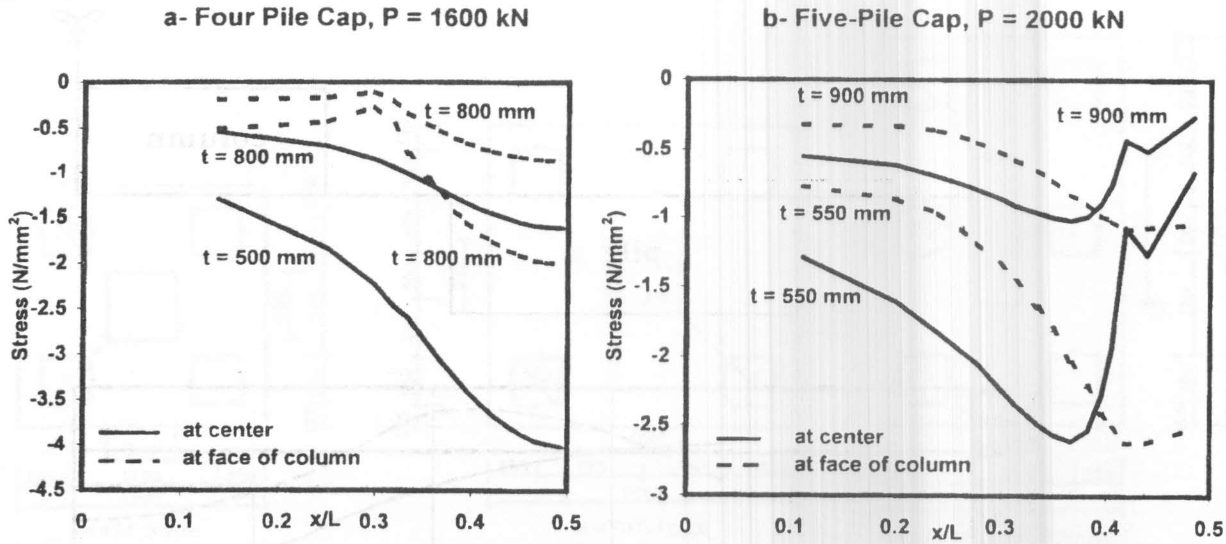

Fig. 4. Variation of flexural stresses along cap width.

Table 1. Summary of caps analyzed.

	Cap thickness, t, mm	Effective depth, d, mm	Shear span, a, mm	a/d	a/d[a]	μ[b] %
Four-pile cap	800	730	424.3	0.58	1.51	0.16
	700	630	424.3	0.67	1.75	0.19
	650	580	424.3	0.73	1.90	0.24
	600	530	424.3	0.80	2.08	0.26
	550	480	424.3	0.88	2.29	0.29
	500	430	424.3	0.99	2.56	0.37
Five-pile cap	900	830	707.1	0.85	1.33	0.17
	800	730	707.1	0.97	1.51	0.19
	750	680	707.1	1.04	1.62	0.21
	700	630	707.1	1.12	1.75	0.25
	650	580	707.1	1.22	1.90	0.33
	600	530	707.1	1.33	2.08	0.43
	550	480	707.1	1.47	2.29	0.58
Six-pile cap	1100	1030	400.0(680.0)[c]	0.39(0.66)[c]	1.07	0.16
	1000	930	400.0(680.0)[c]	0.43(0.73)	1.18	0.15
	900	830	400.0(680.0)[c]	0.48(0.82)	1.33	0.17
	800	730	400.0(680.0)[c]	0.55(0.93)	1.51	0.22
	700	630	400.0(680.0)[c]	0.63(1.08)	1.75	0.35
	650	580	400.0(680.0)[c]	0.69(1.17)	1.90	0.45
	600	530	400.0(680.0)[c]	0.75(1.28)	2.08	0.58

a) L = distance between centerlines of piles = 1100 mm
b) μ = percentage of bottom longitudinal steel in each direction = (area of reinforcement /m) / 1.0 * d
c) shear span for the nearest pile- value in parenthesis is the shear span for other piles

Table 2 Results from the elastic FE analysis and conventional rigid beam method.

Cap thickness t (mm)	Two-way shear v_p (N/mm^2)	One-way shear[a] ECP-98 v_u[b] (N/mm^2)	One-way shear[a] ACI-99 v_u[c] (N/mm^2)	Bending moment[a] (KN.m/m)	Elastic finite element analysis: Shear stress[d] (N/mm^2)	Elastic finite element analysis: Bending moment[e] (kN.m/m)
a- Four-pile caps at P = 2400 kN						
800	0.67	0.29 (0.26)	0.37	127.8	0.42 (0.53)	88.7 (137.6)
700	0.84	0.48 (0.36)	0.47	141.7	0.59 (0.69)	87.1 (134.7)
650	0.96	0.61 (0.42)	0.53	148.1	0.69 (0.78)	85.8 (132.9)
600	1.10	0.75 (0.50)	0.61	154.5	0.82 (0.91)	84.8 (130.7)
550	1.28	0.93 (0.62)	0.71	160.8	0.95 (1.06)	83.4 (128.3)
500	1.50	1.15 (0.76)	0.84	166.8	1.14 (1.27)	81.7 (125.3)
b- Five-pile caps at P = 3000 kN						
900	0.52	0.48 (0.27)	0.16	183.9	0.41 (0.46)	141.5 (217.8)
800	0.64	0.65 (0.35)	0.21	197.3	0.56 (0.72)	140.7 (215.0)
750	0.72	0.75 (0.41)	0.00f	204.0	0.62 (0.96)	138.6 (212.7)
700	0.81	0.85 (0.48)	0.11 f	210.6	0.74 (1.20)	138.3 (210.0)
650	0.92	0.92 (0.56)	0.25 f	217.0	0.84 (1.32)	136.4 (206.4)
600	1.05	1.01 (0.67)	0.42 f	223.3	0.95 (1.41)	134.7 (203.0)
550	1.21	1.11 (0.82)	0.62 f	229.3	1.06 (1.43)	132.1 (198.3)
c- Six-pile caps at P = 3600 kN						
1100	0.51	0.39 (0.20)	0.18	124.5	0.34 (0.51)	128.5 (200.4)
1000	0.59	0.53 (0.26)	0.25	139.2	0.44 (0.61)	127.2 (199.2)
900	0.71	0.71 (0.33)	0.29	153.7	0.57 (0.76)	126.4 (198.3)
800	0.86	0.91 (0.43)	0.37	168.0	0.76 (1.13)	124.4 (195.1)
700	1.07	1.06 (0.57)	0.48	182.0	0.96 (1.34)	120.0 (190.5)
650	1.21	1.15 (0.67)	0.48 f	188.9	1.07 (1.35)	119.7 (187.3)
600	1.38	1.26 (0.81)	0.70 f	195.8	1.17 (1.95)	113.2 (184.0)
Notes:	v_{cp} = 1.58	v_c = 0.78	v_c = 0.84-1.07	At the face of the column	At d/2 from the face of the column	At the face of the column

[a] the column load is distributed to the mid-depth of the cap according to the rigid beam method.
[b] values in parenthesis are obtained at the column face without distribution of column load and the shearing force is multiplied by a/2d, where a = 424.3 mm for four-pile caps, 707.1 mm for five-pile caps, and 680.0 mm for six-pile caps (see Fig. 1).
[c] the critical section is at the face of the column.
[d] average values along cap width- values in parenthesis are maximum values of shear stresses.
[e] average values of moments along cap width- values in parenthesis are maximum values of moments- moments recorded for six-pile caps are in the short direction of the cap.
f the critical section is at d from the column face.
v_{cp} and v_c = nominal values of shear stresses provided by concrete according to codes of practice.

section obtained from the FE analysis were less than those obtained using the rigid beam method by 30 to 51% for 4-pile caps, 23 to 42% for 5-pile caps, and 0 to 42% for 6-pile caps. However, the maximum values of moments obtained from the analysis were very close to those obtained using the rigid beam method, for L/d = 1.75.

These results suggest that when designing 4 and 5 pile caps, the calculated flexural reinforcement could be reduced by at least 50% in the outer portions of the cap.

iii-Figure 5 shows the variation in the flexural stresses for two cases of 6-pile caps along short and long directions of the caps. It can be observed that the finite element analysis predicted negative moments (tension top) away from the column and extended to the free end of the caps. Values of negative moments were higher for caps with low rigidity. With the assumption of rigid caps, the rigid beam method does not predict such moments.

iv-Table 2 shows that the average values of one-way shear along cap width as obtained by the FE analysis at the critical section defined by ECP-98 (at d/2 from the face of the column) agreed well with the values predicted by the rigid beam method. Along the critical section the maximum values of shear stresses occurred at the piles edge. The predicted values of shear stresses (either by the analysis or by the rigid beam method) were less than the value permissible by the ECP-98 up to L/d ratios of 2.08, 1.75, and 1.51 for 4-pile caps, 5-pile caps, and 6-pile caps respectively. As shown in Table 2, the introduction of the effect of a/2d ratio, as recommended by the ECP-98 on the predicted stresses yielded shear stresses less than the permissible value in most cases, even if the critical section is taken at the column face. The shear stresses, calculated according to ACI 318-99 (i.e. critical section at the face of the column) and considering the distribution of column load through cap thickness, were less than the permissible values in all cases.

Figures 6-a and 6-b show the distribution of transverse shear stresses along different sections of 4 and 5-pile caps, respectively. The figures indicate that very high values of shear stresses occurred at both the pile corner and at the column edge for all cases of 4-pile caps and 5-pile caps. For the case of 6-pile cap and in the short direction, high values of shear stresses were localized in the area between the column and the closest pile, as shown in Fig. 6-c.

v- For all the cases studied, the values of two-way (punching) shear at the critical section (d/2 from the face of the column) as predicted by the rigid beam method were less than the value recommended by the codes.

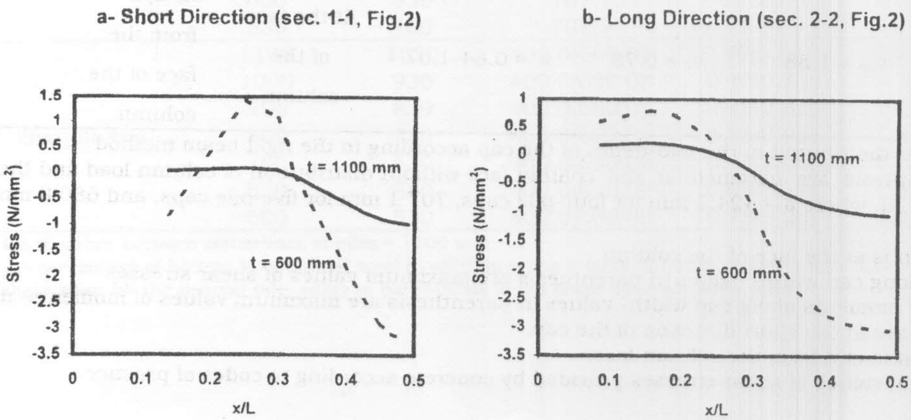

Fig. 5. Variation of flexural stresses, for six-pile cap, p =2400 kN.

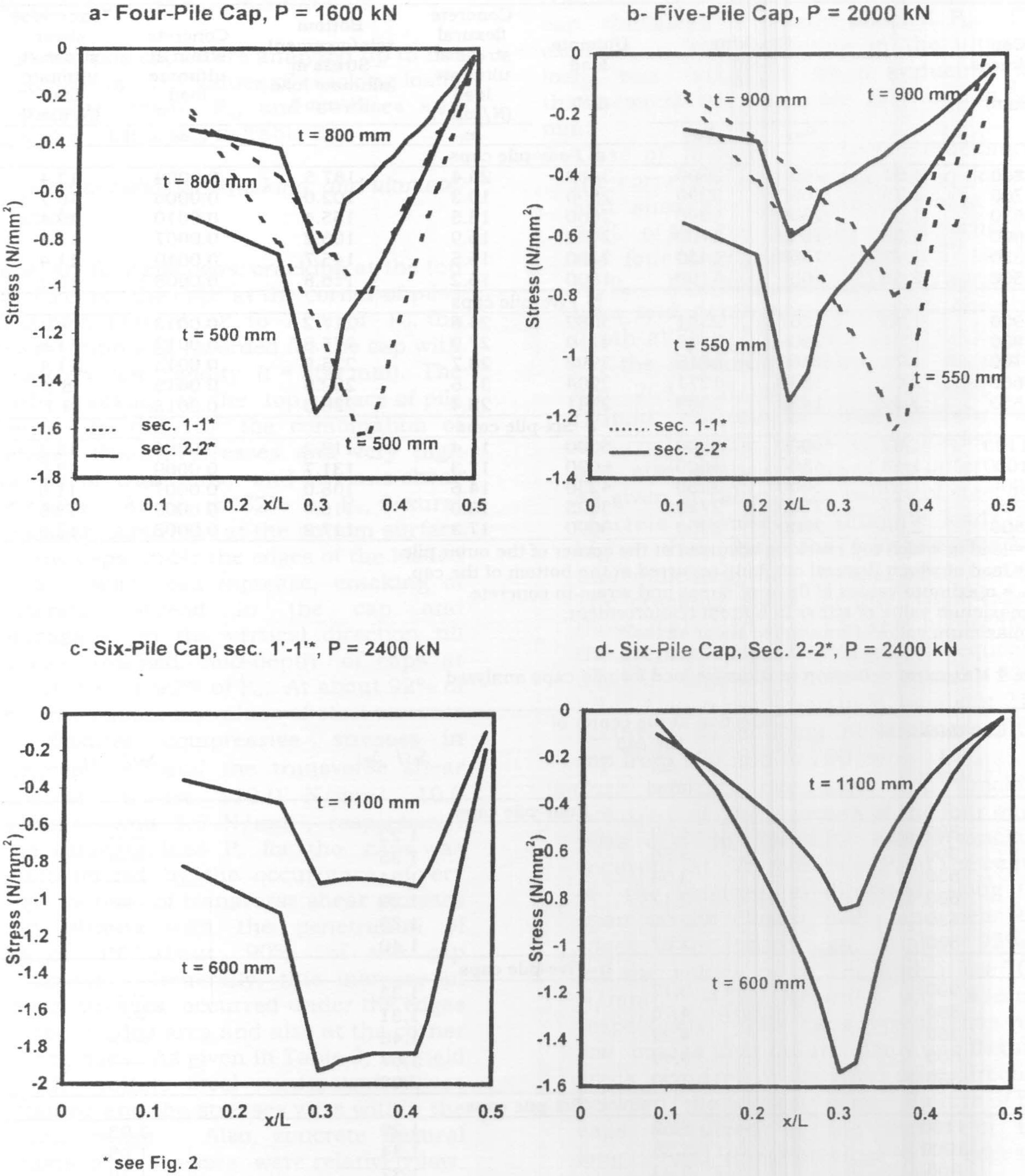

Fig. 6. Variation of transverse shear stresses.

Table 3 Results from the non-linear finite element analysis

Cap thickness t (mm)	L/d	Cracking load (kN)		Ultimate load P_u (kN)	Concrete flexural stress at ultimate load (N/mm²)	Bottom reinforcement Stress at ultimate load (N/mm²)	Concrete strain at ultimate load	Transverse shear stress at ultimate load (N/mm²)
		P_{cr1}	P_{cr2}		σ_c	f_s	ε_c	τ
				a- Four-pile caps				
800	1.51	2400	2800	3400	20.4	187.5	0.0009	13.4
700	1.75	2000	2290	2800	10.3	102.0	0.0006	16.2
650	1.90	1800	1990	2600	13.5	155.4	0.0010	19.4
600	2.08	1600	1700	2400	16.9	106.2	0.0007	11.1
550	2.29	1350	1430	2100	16.5	145.7	0.0010	11.4
500	2.56	1050	1190	1800	16.2	116.8	0.0008	11.1
				b- Five-pile caps				
900	1.33	3267	2541	5082	24.6	217.2	0.0013	9.1
800	1.51	2904	2178	4719	27.9	215.8	0.0013	11.8
700	1.75	2360	1815	3993	28.7	225.5	0.0015	11.4
600	2.08	1634	1271	2904	31.6	282.9	0.0019	8.3
550	2.29	1452	1089	2541	28.4	228.0	0.0015	9.1
				c- Six-pile caps				
1100	1.07	5000	5500	6500	11.4	118.2	0.0006	13.2
1000	1.18	4500	4500	6000	17.3	131.7	0.0009	13.5
800	1.51	3000	3250	4250	14.6	108.0	0.0007	11.6
700	1.75	2500	2750	3625	15.0	116.4	0.0008	13.6
600	2.08	2000	2250	3000	17.3	117.8	0.0008	12.6

P_{cr1} = load at which top cracking occurred at the corner of the outer pile.
P_{cr2} = load at which flexural cracking occurred at the bottom of the cap.
σ_c, ε_c = maximum values of flexural stress and strain in concrete.
f_s = maximum value of stress in bottom reinforcement.
τ = maximum value of transverse shear stress.

Table 4 Maximum deflection at ultimate load for pile-caps analyzed

Cap thickness t (mm)	Deflection at the center of the cap Δ_{c1} (mm)	$\Delta_{c1}/\ \Delta_{p1}$	$\Delta_{p2}/\ \Delta_{p1}$
	a- Four-pile caps		
800	4.49	1.22	-----
700	3.56	1.25	-----
650	3.54	1.34	-----
600	3.17	1.29	-----
550	2.96	1.38	-----
500	2.57	1.40	-----
	b- Five-pile caps		
900	5.17	1.33	-----
800	4.76	1.31	-----
700	4.22	1.40	-----
600	3.49	1.67	-----
550	3.79	2.08	-----
	c- Six-pile caps		
1100	5.62	1.29	1.03
1000	5.36	1.34	1.05
800	3.83	1.37	1.10
700	3.62	1.56	1.18
600	3.21	1.71	1.26

Δ_{p1} = deflection at outer piles.
Δ_{p2} = deflection at closest pile for six-pile caps.

3.2 Results from the non-linear analysis

Sixteen pile caps were analyzed up to the ultimate loads. The values of cracking loads P_{cr}, ultimate loads, P_u, and stresses and strains at P_u, are given in Table 3.

3.2.1 General behavior, cracking, and ultimate loads

i- *Case of four pile caps:* cracking at the top surface of the caps at the corner of piles occurred first at 58 to 71% of P_u, the lowest ratio was recorded for the cap with relatively low rigidity (t = 500 mm). The early cracking at the top surface of pile corner is due to the combination of tensile flexural stresses and very high values of transverse and in-plane shear stresses. At 66 to 82% of P_u, flexural cracking occurred at the bottom surface of the caps under the edges of the loaded area. With load increase, cracking of concrete spread in the cap and propagated in the vertical direction till cracks reached mid-depth of caps at about 85 to 92% of P_u. At about 92% of P_u, the maximum values of steel stresses f_s, flexural compressive stresses in concrete σ_c, and the transverse shear stresses τ, are 110.0 N/mm², 10.0 N/mm², and 5.5 N/mm², respectively. The ultimate load P_u for the caps was characterized by the occurrence of very high values of transverse shear stresses and strains with the penetration of cracks to about 90% of the cap thickness. Generally, this increase in shear stresses occurred under the edges of the loaded area and also at the corner of the piles. As given in Table 3, no yield in the bottom steel reinforcement was obtained and the stresses were within the elastic range. Also, concrete flexural strains and stresses were relatively low. The results indicate that failure of caps was by one-way shear.

Generally, the loads at which bottom cracking occurred in all the caps, except those with L/d > 2.0, were higher than the average design working load frequently used in practice which is equal to 1600 kN. The results of the ultimate loads indicate that the cap thickness t greatly affected P_u. For example, a 24% reduction in the ultimate load was obtained when reducing the thickness of the cap from 800 mm to 650 mm.

ii- *Case of five pile caps:* bottom cracking at the corners of central piles, due to flexural and shear stresses, occurred first at 43 to 50% of P_u. Top cracking at the corners of the four outer piles occurred at 57 to 64% of P_u. Cracking continued to spread in plan and depth till it reached the mid-depth at 70 to 85% of P_u. At about 92% of P_u, the maximum values of f_s, σ_c, and τ are 204.0 N/mm², 25.9 N/mm², 7.9 N/mm², respectively. Failure of the caps occurred at the edges of the loaded area and also at the corners of the outer piles. As given in Table 3, the values of steel and concrete stresses at ultimate loads are higher than those for four-pile caps and this indicates that flexural being critical. The cracking loads for the caps, except those with L/d > 1.75, were higher than the average design working load frequently used in practice which is equal to 2000 kN. Also, a reduction in P_u by about 21% occurred by reducing the thickness of the cap from 900 mm to 700 mm.

iii-*Case of six pile caps:* Top cracking occurred at the corners of the four outer piles at 67 to 77% of P_u. Bottom cracking occurred at 75 to 85% of P_u at the center of the longitudinal direction (along the span of the closest piles) and near the edges of the loaded area. At about 92% of P_u, the values of f_s, σ_c, and τ are 70.0 N/mm², 9.7 N/mm², 9.6 N/mm², respectively. The caps typically has very few cracks near failure. Only one flexural crack occurred in the short span direction between the closest piles. Failure of the caps occurred at the center of the longitudinal direction close to the edges of the loaded area and also at the corner of the closest piles. The concrete compressive stresses and strains at failure were relatively low. The cracking loads for the caps, except those with L/d > 2, were higher than the average design working load frequently used in practice which is equal to 2400 kN.

3.2.2 Effect of cap thickness on ultimate load

Figure 7 shows the relationship between the non-dimensional strength parameter (P_u/design ultimate load) and L/d ratio for the sixteen pile caps studied. The design ultimate load was taken as 1.5 times the average working load, generally used for the design of pile caps. The values of the design ultimate load were taken as 2400 kN for 4-pile caps, 3000 kN for 5-pile caps, and 3600 kN for 6-pile caps. The figure indicates that for 4 and 5-pile caps with $L/d \leq 2$, the ultimate load obtained from the analysis was higher than the design ultimate load. However, for 6-pile caps such L/d ratio was about 1.75. Introducing a factor of safety of 1.25 to the design ultimate load, and assuming $d = 0.9\,t$. It is concluded from the results plotted in Fig. 7 that the cap thickness should not be less than 0.65L, 0.6L, and 0.77L for the case of 4, 5, and 6-pile caps, respectively.

3.2.3 Deflections

Table 4 gives the maximum values of deflections obtained from the FE analysis at ultimate loads, whereas Fig. 8 shows the load-central deflection relationship for the caps studied. Figure 8 indicates that for all the caps studied, load-deflection relationship is approximately linear up to 80-90% of P_u. Also, the effect of pile rigidity (thickness) on the values of deflection started to appear at about 95% of the average design working load of the pile group. For all cases studied, the maximum deflection at P_u was only 1/1000 of the cap thickness (approximately 1/100 of the distance between piles).

Table 4 indicates also that at ultimate loads of the five pile caps, the ratio of the load (which is linearly proportional to the pile deflection) carried by the central pile to that carried by each of the other outer piles is varied between 1.33 to 2.08 for t = 900 mm (L/d = 1.33) to t = 550 mm (L/d = 2.29),

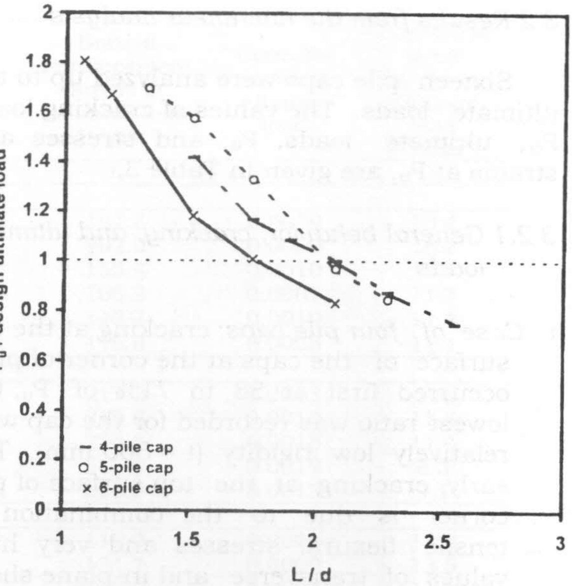

Fig. 7. Effect of cap thickness on the ultimate load capacity

respectively. Generally, up to 80% of the average design working load of the pile group, no large difference in pile loads was recorded for all cases studied.

However, for the case of 6-pile caps the difference in the load carried by the closest piles and that carried by other piles was ranged between 3 and 26%. It should be noted that in the present study, columns for the case of 6-pile caps were taken as rectangles and if a square loaded area were considered a larger difference in loads carried by piles may be obtained as recorded experimentally by Adebar et al. [3].

4. Conclusions

Based on the results obtained in this study, the following general conclusions can be made:

i- All caps having L/d ratio less than 1.75 remained uncracked up to the average design working load of the pile group. Therefore, the use of a linear elastic stress analysis is relevant.

ii- The elastic analysis indicated that for five-pile caps having one pile directly under the column, the ratio of the load carried by the central pile to that carried by each

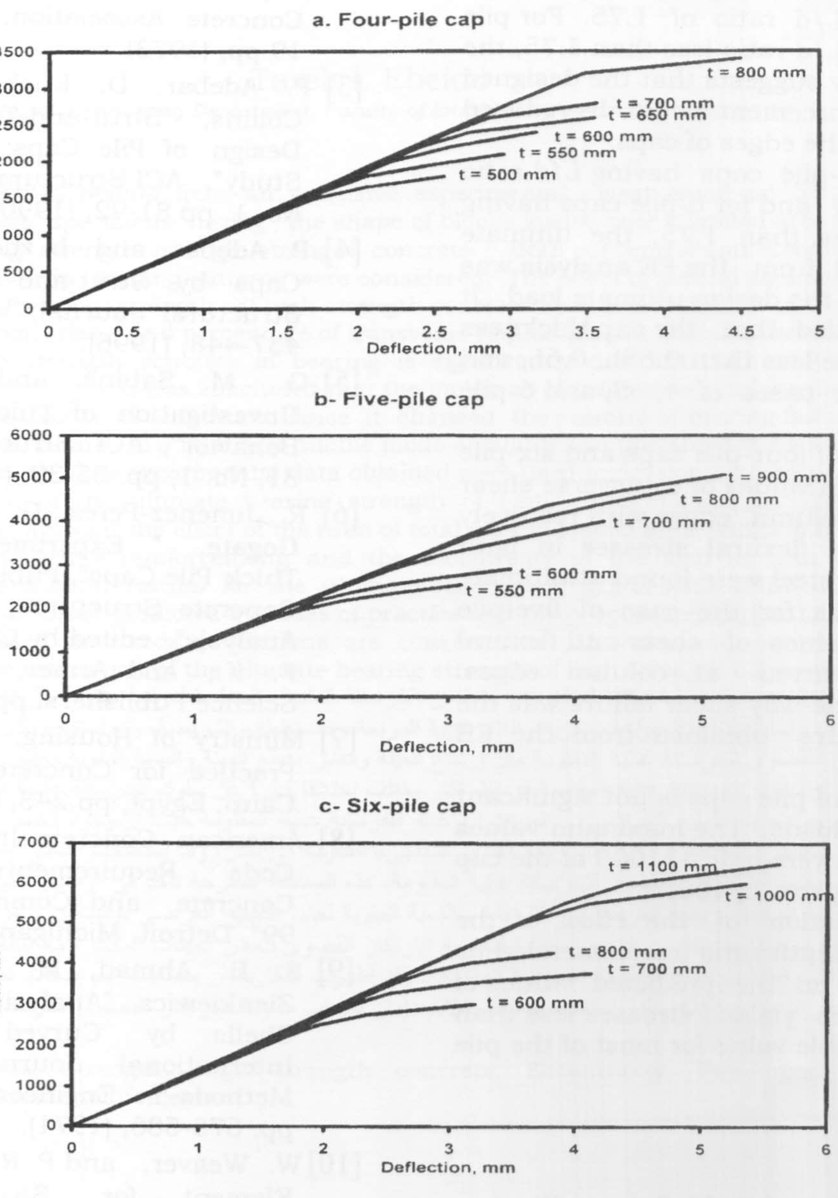

Fig. 8. Load-central deflection relationships

of the four outer piles ranged between 1.06 and 1.14, depending on the cap thickness. However, at the ultimate load, such ratio ranged between 1.03 and 2.08. For six pile caps, loaded through rectangular columns, the ratio of the load carried by each of the two closest piles to that carried by each of the four outer piles ranged between 1.028 and 1.09, as obtained from the elastic analysis. At the ultimate load, the above ratio ranged between 1.04 and 1.26.

iii-In the rigid beam method it is assumed that pile caps behave as wide beams. However, it was found from the present FE analysis that only a concentrated zone of the cap under the column resists significant load. At the critical section and along cap width, the maximum values of moments were very close to those obtained using the rigid beam method, for L/d ratio of 1.75. For pile caps having L/d ratio less than 1.75, the present study suggests that the designed flexural reinforcement could be reduced by 50% near the edges of caps.

iv- For 4 and 5-pile caps having L/d ratio less than 2.0 and for 6-pile caps having L/d ratio less than 1.75, the ultimate load obtained from the FE analysis was higher than the design ultimate load. It is recommended that the cap thickness should not be less than 0.65L, 0.6L, and 0.77L for the cases of 4, 5, and 6-pile caps.

v- In the case of four-pile caps and six-pile caps very high values of transverse shear stresses at column edges with relatively low values of flexural stresses in both concrete and steel were found at ultimate loads whereas for the case of five-pile caps, high values of shear and flexural stresses occurred at column edges. Generally, one-way shear failure was the type of failure obtained from the FE analysis.

vi- Deformation of pile caps is not significant at ultimate loads. The maximum values of deflection were only 1/1000 of the cap thickness (approx. L/100).

vii-The consideration of the effect of the shear span/depth ratio (recommended by the ECP-98) on the predicted values of shear stresses yielded stresses less than the permissible value for most of the pile caps studied.

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