Experimental study on load bearing behavior of large-scaled caps with pile groups

Guo Chao[1], Lu Bo[2], Gong Weiming[1], Qiu Hongxing[1]
(1. Department of Civil Engineering, Southeast University, Nanjing 210096, China;
2. Quality Supervision Station of Rizhao Planning and Urban Construction Committee, Shandong, Rizhao 276826, China)

Abstract: The objective of this investigation was to study the behavior of deep pile caps and the ultimate load-carrying capacity. Four 1/10 scaled models of nine-pile caps were cast and tested on vertical loads to failure. The destruction shapes of pile caps, the correlation between load and displacement, and the internal stresses were analyzed systematically. The results demonstrated that the failures of all the four models are resulted from punching shear; the internal flow of the forces in nine-pile caps can be approximated by "strut-and-tie" model. Furthermore, the failure loads of these specimens were predicted by some of the present design methods and the calculated results were compared with the experimental loads. The comparative results also indicated that the "strut-and-tie" model is a more reasonable design method for deep pile caps design.
Key words: large-scaled caps with pile groups; punching shear; strut-and-tie model; failure load

1 Introduction

At present, the general methods[1,2] used to design of pile caps in China can be summarized as follows. First of all, the plan dimensions of a pile cap were determined by the quantity of piles and forms of pile layout. Secondly, the required thickness of pile cap is predicted by engineering experience in advance. Finally, the pile cap was seen as a general flexural member whose bearing capacity of bending, punching and shearing will be calculated respectively. However, owing to the low tensile strength of concrete, the bearing capacity of pile cap is generally controlled by punching shear, the required thickness of pile cap increasing with the upper loads. The leading cause of the above problem is that the failure mode and load-transferring mechanism of pile caps have not been comprehended clearly. Most of these caps are deep and behave unlike flexural member. The results of experimental studies[3-6] have demonstrated that a strut-and-tie model is a more appropriate design method for deep pile cap than the general methods "flexural member" approach because it better presents the actual behavior of pile cap. Unfortunately, all of the research works having been done on pile caps were merely limited to six plies. What is the actual behavior of deep pile cap with multi-lines and multi-rows of piles is the focus of the present study.

2 Model design and fabrication

Specimens for testing are scaled models, since full-scale pile caps would be not only expensive but also unmanageable in available facilities in the laboratory. Four 1/10 scaled nine-pile caps with identical plan dimensions are fabricated to simulate the practical bridge engineering. These specimens will be labeled from CTA to CTD.

The available experimental investigation[7] indicated that the width-to-depth ratio ($\lambda = a/h_0$) can be used for defining the deep pile caps. If $a/h_0 \leqslant 2$, the pile cap belongs to thick pile cap (where a represents the horizon distance between the column centre and the farthest pile centre, h_0 represents the effective depth of pile cap). All of the width-to-depth ratios of these specimens which are given in Table 1 are smaller than 1.84, so the four specimens can be classified into the deep pile caps.

The details of these specimens are shown in Fig. 1, Table 1 and Table 2. The arrangement pattern of piles is 3×3, the column dimensions are 300 mm × 300 mm × 150 mm, the piles are reinforced concrete circular cylinders with 150 mm in diameter and 330 mm in height, which are embedded in pile cap 30 mm. Concrete used for the caps is premixed C20 concrete. Columns and piles are cast in the same batch and cured for 28 days in the standard curing room.

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Adopt ϕ10 main steel bar and ϕ6 round steel bar for piles while adopt ϕ16 main steel bar and ϕ8 round steel bar for columns.

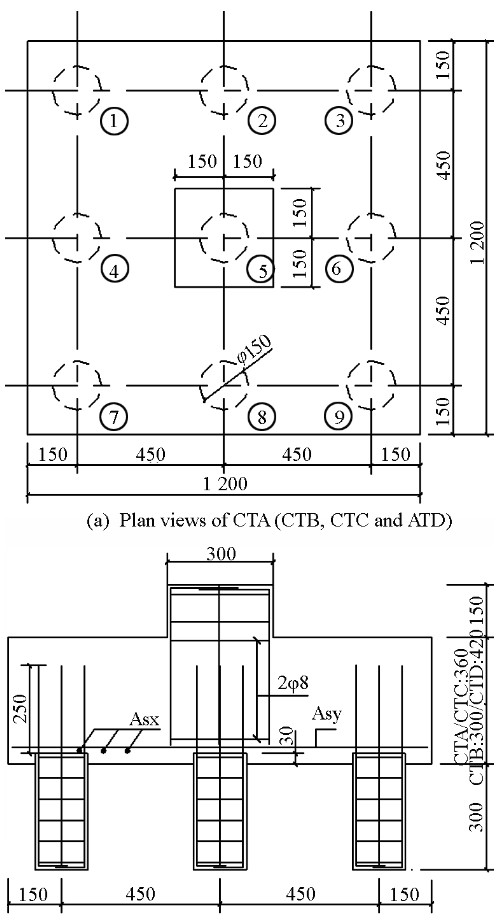
Table 1 Details of pile caps

Label	H/h_0/mm	a/mm	a/h_0
CTA	360/307	450	1.46
CTB	300/245	450	1.84
CTC	360/306	450	1.47
CTD	420/375	450	1.20

Table 2 Results of material test

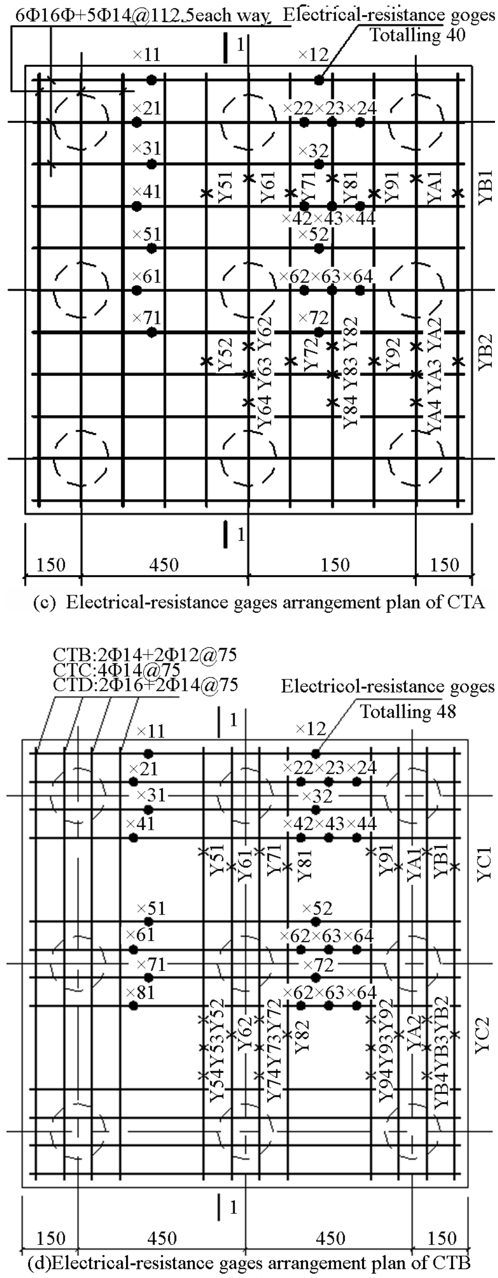
	Spec.	f_{yk}/MPa	f_{tk}/MPa	E_s/MPa
Reinforcement	Φ12	372	548	2.01×10^5
	Φ14	369	558	2.03×10^5
	Φ16	374	548	2.03×10^5
	Label	$f_{cu,k}$/MPa	f_{ck}/MPa	f_{tk}/MPa
C20 concrete	CTA	31.75	21.27	2.08
	CTB	29.15	19.53	1.97
	CTC	32.89	22.04	2.12
	CTD	31.35	21.00	2.06

Electrical-resistance gages are installed at various locations on selected reinforcements in the four specimens as shown in Fig. 1. Displacement transducers are used to detect displacements (deflections) at various positions on the specimens which include the bottom of each pile, the column top and the center point of 1/4 region at the bottom of pile cap.



(a) Plan views of CTA (CTB, CTC and ATD)

(b) Cross-section 1-1



(c) Electrical-resistance gages arrangement plan of CTA

(d)Electrical-resistance gages arrangement plan of CTB (CTC and CTD)

Unit:mm

Fig. 1 Constructional details of nine-pile caps

A 3 500 kN testing frame is used to apply vertical compressive load onto the specimens. The total loads applied to the column are measured using oil gauge. Piles are supported on steel pedestals, which in turn are supported on rubber pads. Piles with different rigidities could be simulated by varying the rigidities of rubber pads, the influence of pile rigidity to the rigidity of pile cap and the distribution of pile loads are analyzed separately in the reference[8] and not presented in this paper. The load on the columns is increased by step; the load increment of each step accounts for

10 % to 15 % of the predicted failure load. All electronic measurement data are automatically recorded by computer.

3 Analysis and discussion of results

3.1 Formation of cracks and failure load

A number of observations were made during the tests. The crack loads and failure loads were given in Table 3. Fig. 2 shows the observed crack patterns of a part of pile caps at failure.

Table 3 Crack loads and failure loads of pile caps

Label	CTA	CTB	CTC	CTD
Crack load/kN	700	650	700	1300
Failure load/kN	1 900	1 800	2 100	3 300

According to the requirement of the general methods aforesaid, the required longitudinal reinforcements were distributed uniformed across CTA. The first crack appeared at the bottom of the cap at approximately 700 kN and extended to the sides promptly. The strains in the reinforcements grew linearly before cracking and suddenly increased as soon as the first crack appeared. Cracks continued to develop though the load increased at a lower rate. Many new cracks developed around piles before the failure load was reached; when the total load reached up to the maximal load of 1 900 kN, the specimen continued to deflect with no further increase in load, a punching shear failure occurred with a square crack pattern within the inside edges of the peripheral eight piles. Removed failed concrete along the failure cracks, a punching cone could be found which extended from the column's outside faces to the inside edges of the piles.

CTB and CTC formed the first crack at the bottom of the cap at 650 kN and 700 kN respectively, have the similar crack load to CTA. This shows that the changed forms of reinforcement layout have hardly any influence on the cracking load of pile caps. As the load continued to increase, diagonal cracks developed among the center pile, corner piles and side piles successively, and then traversed progressively to the sides. When the load increased from 50 % to 80 % of the failure load, the primary cracks kept on extending, but few new cracks developed, at the same time, the strain values of reinforcements were growing fast. The phenomena aforesaid indicated that reinforcements gave full play to tension. When the load increased to be the failure load, a part cracks on the sides of pile cap extended near the top surface, a large number of radial cracks kept on developing between corner pile and side pile at the bottom of the pile cap which indicated the failure of compression struts. The failure load reached in company with several splitting cracks developed on the top surface, one of the distinctive marks of punching shear. There was a horizontal crack along full length of one side, which indicated that the punching shear gave rise to anchoring failure of reinforcement and sag of protecting coating.

CTD, which was designed with the deepest depth and largest amount of reinforcements of the four specimens failed at a maximum load of 3 300 kN. The failure load was 83 % higher than the CTB. The first crack appeared at the bottom of pile cap CTD at the load of 1 300 kN, 100 % higher than the crack load of CTB. It is indicated that increment in thickness of pile cap and reinforcement quantities can effectively heighten the crack load and failure load of the cap. At the failure load of 3 300 kN, the finger of oil gauge sprung back, side piles and corner piles unloaded, the column load transferred directly to the centre pile, several diagonal cracks on the top surface burst into view. It can be seen from the failure specimen (Fig. 2) that there was not consecutive crack around the peripheral eight piles and horizontal crack on the side of pile cap but plenty of radius cracks around each corner pile, the pile cap presented a punching shear by corner piles.

3.2 Relationships analysis between load and displacement

Each of these nine-pile caps can be divided into four parts by piles under the cap, and the relative deflections can be obtained by displacement transducers at cap bottom. The observed load-deflection relationships for the four specimens are shown in Fig. 3, thereinto, the deflection of each pile cap is the average deflection in the regional centers of four parts. It can be seen from the figure that pile cap deflection increasing with the decreasing of depth in the same level of load. Before cracking, there was no obvious turning point in the load-deflection curve. During the loading process, there was no distinct horizontal segment in any load-deflection curve. However, the deflection increased suddenly without any premonition, indicated representative brittle failure. It was hardly to form consecutive "plastic hinge line[9]" in pile cap even though reinforcement in some positions yielded, which was different form the ordinary flexural member. Therefore, it will not be appropriate that applying the "plastic hinge line" theory to design of pile caps.

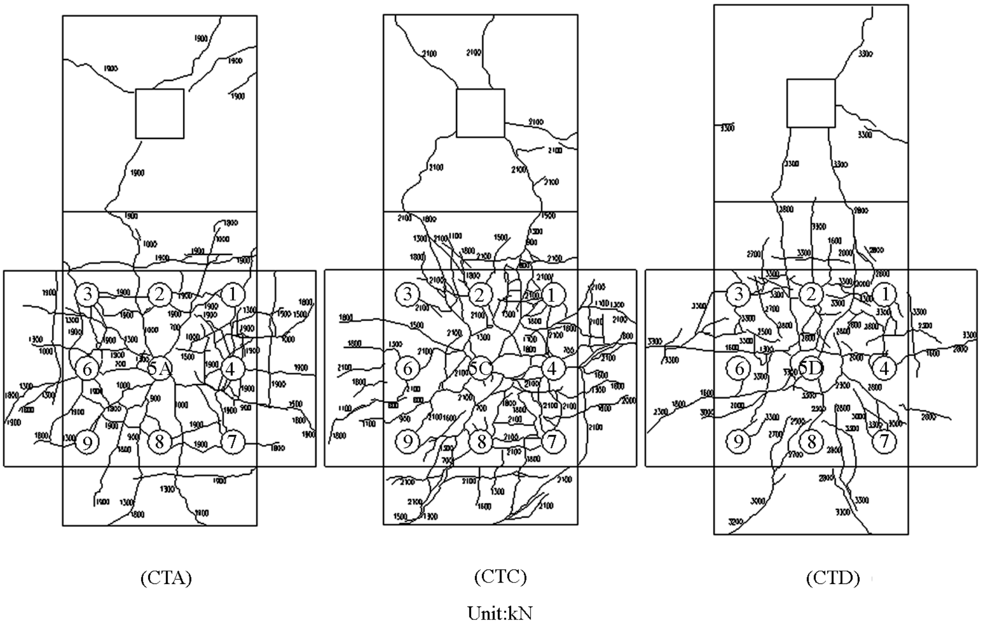
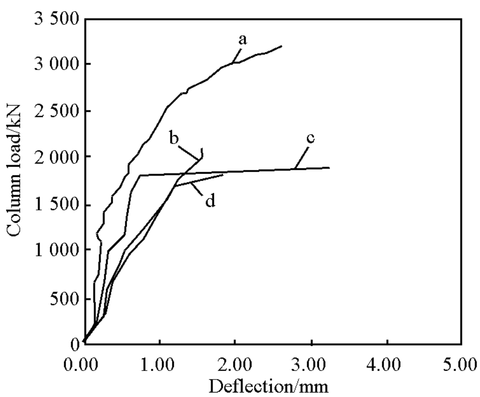


Fig. 2 Expanded view of specimen cracks distribution



a—CTD; b—CTC; c—CTA; d—CTB

Fig. 3 Load-deflection relationships for specimens

3.3 Strain analysis of reinforcement

Strains of longitudinal reinforcement were measured at various column loads. Fig. 4 shows a typical example of load-reinforcement strain variations. Before cracking, all of the reinforcement strains increased at a row rate and the corresponding load-strain curves were slow ascending straight segments, tension at the bottom of pile cap was resisted by reinforcement and concrete together. There were obvious turning points in load-strain curves at the crack load. After cracking, the increment speed of reinforcement strain was far beyond the speed before cracking, and tension at the bottom of pile cap was resisted primarily by reinforcement.

The strain trend of reinforcement varied with the thickness of pile cap, which was shown in Table 4. For a given load, the strain increased with the decreasing of thickness. In other words, the crack load increased while the strain increment speed slowed down. It can be inferred that after cracking of the concrete which at the bottom of the deep pile cap, the strain increment speed is accelerated. The concrete at the cracking area stops working while the reinforcement resists the tension. But the uncracked concrete above the reinforcement will continue to share part of the tension, and with the thickening of the pile cap, the contribution proportion of concrete is also increasing. It's conservative when only considering the influence of reinforcement while neglecting the concrete on tension resistance.

Table 4 Strains varied with the thickness of pile cap unit: με

Load/kN	300	500	600	700	900	1 000	1 200	1 300	1 500	1 600	1 700
BX42	36	66	84	152	855	1 138	1 480	1 614	2 027	3 080	5 356
CX42	18	38	54	74	169	318	691	877	1 056	1 126	1 244
DX42	14	29	36	44	58	66	89	108	154	194	313

As far as the same reinforcement was concerned, strains at various locations tend to a uniform value and kept the same amplitude, presented a tension member along the full length. The strain variation was contrary to the behavior of flexural member in which the tension force changed along the length of longitudinal reinforcement. The tension force in reinforcement conforms to the distinguishing feature of strut-and-tie models.

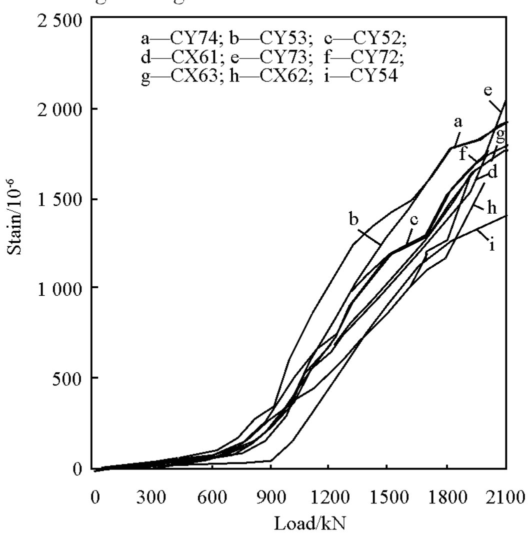


Fig 4 Load-reinforcement strain variations of CTC

4 Comparison of observed strength with the present design methods

In general, the design methods for pile caps can be divided into two kinds. The first kind sees the pile cap as a general flexural member and the bearing capacity of flexure, punching and shear will be calculated according to the code for design of concrete structure. The other kind is suggested to be analyzed by adopting strut-and-tie model according to the spatial force suffering character of deep pile cap. The internal load path in cracked reinforced concrete is approximated by an idealized truss. Zones of concrete with primarily unidirectional compressive stresses are modeled by compression struts and tension ties are used to model the principal reinforcement[10].

The failure loads of these specimens were predicted using the first kind of method (GB50007-2002[2] and ACI318-02[11]) and the second kind of method (proposed by Lu Jianfeng in Ref. [7]). Results of the analysis with experimental loads are present in Table 5.

Table 5 Comparison of predictions and experimental failure loads

Label	Predicted loads/kN			Experimental load/kN	Exp./Pred.		
	Ref. [2]	Ref. [7]	Ref. [11]		Ref. [2]	Ref. [7]	Ref. [11]
CTA	1 337	1 900	947	1 900	1.421	1.000	2.006
CTB	841	1 201	725	1 800	2.140	1.499	2.483
CTC	1 259	1 789	906	2 100	1.668	1.174	2.318
CTD	1 890	2 648	1 150	3 300	1.746	1.246	2.870

All of the three methods are safe and credible for design of pile caps. However, GB50007-2002 and ACI318-02 predictions of failure load are excessively conservative for deep pile caps. In ACI318-02 the maximum ratio of experimental load to predicted load is 2.870, indicating that some modification of the theory shall be made with more experimental work. The design method proposed by Lu Jianfeng introduced strut-and-tie models that consider the behavior of internal compression struts in pile cap, the failure load can be figured out by limiting the bearing stress to avoid splitting in concrete compression struts. The analytical results indicated to a certain extent that the general methods are not suitable for design of deep pile caps, strut-and-tie models can be adopted for the design.

5 Conclusions

All of the four pile caps are brittle specimens and failed in punching shear. Strut-and-tie models were found to be reasonable for design of deep pile caps, which have been supported by the available test data. For a given quantity of steel, bunching the longitudinal reinforcement resulted in higher bearing capacities than distributing in a uniform form. The longitudinal reinforcement shall be concentrated over the piles to heighten the bearing capacity. In addition, to prevent pile caps from anchoring failure, the bottom horizontal reinforcement shall satisfy the anchoring requirement; the anchoring starting point can be calculated from the centre of exterior piles. However, the crack loads were not significantly influenced by whether the reinforcement was distributed in bunch or in a uniform form.

The increment of crack and failure loads of CTD indicated that the depth has a strong influence on the bearing capacity of pile cap. The uncracked concrete above the reinforcement shares part of the tension and the contribution proportion of concrete is increasing with the thickness of pile cap. Therefore, the concrete contribution to tensile resistance shall not be neglected before cracking.

In reinforced concrete pile caps, there are some key factors contributed to the bearing capacity, such as depth, amount of reinforcement and reinforcement arrangement. There are large variations of failure loads predicted by the present design methods of pile caps. The failure load of a deep pile cap is considerably underestimated using the *flexural member*. Strut-and-tie models conform to the internal flow of the forces in deep pile caps and can be used for design.

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Author

Guo Chao, male, was born in 1984. He graduated from Southeast University and now is a doctor student in Geotechnical Engineering at Southeast University, Nanjing, China. Mr. Guo has published over 5 papers. His current research is deep pile caps, composite pile foundation constructed in top-down order etc. He can be reached by E-mail: guochao422@163.com

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