Site test study of pile-soil stress ratio of composite foundation in karst area

Jianqiang Han 1, Weike Li1,2*, Junguang Huang 1 and Jie Cui 2

1 Guangzhou Design Institute, Guangzhou, Guangdong, 510620, China
2 Guangzhou University, Guangzhou, Guangdong, 510620, China
*Corresponding author’s e-mail: liweike@aliyun.com

Abstract. Aiming at the special geological conditions in Karst areas, the pile-soil stress ratio of composite foundation is analyzed. According to carry out the field test of composite foundation in Karst area, soil pressure boxes are arranged to measure the stress of piles and soil at the same time. As the load increased, the variation trend of the pile-soil stress ratio of composite foundation is analyzed. On the basic, the pile-soil stress ratio is compared with non-karst area. The test result showed that both the pile-soil stress ratio of PHC piles and high pressure jet grouting piles increase. The increase of the PHC piles is more obvious. The pile-soil stress ratio of PHC piles is larger than high pressure jet grouting piles. The main reason is that the strength of cement-soil mixed piles is lower. The high pressure jet grouting piles plays an important role in improving soil bearing capacity between piles and supporting the side abutment pressure of PHC piles in composite foundation. It is shown that the stress concentration is more obvious in karst area.

1. Introduction
Composite foundation is extensively adopted by construction projects in karst areas due to its shortened duration, excellent load-bearing performance and low cost [1-3]. As one of the main study objectives in design of composite foundation, pile-soil stress ratio has been much studied. Minghua Zhao et al. [4] deducted the quantified formula for calculating pile-soil stress ratio in non-karst areas. Shidong Ma [5] and Xuqun Zhang et al. [6] performed site tests on pile-soil stress ratio of composite foundation with various types of piles at non-karst areas and obtained much data. However, in karst areas, karst (soil) caves may exert significant reduction effect on load-bearing capacity of the piles and soil between piles [7], and the difference relative to pile-soil stress ratio of piles in normal ground shall be identified. Zhiyong Xia [8] introduced practical application of composite foundation with CFG piles in karst area of Guilin and indicated that differing from the application in normal area, application of composite foundation with CFG piles in karst area is bothered by soft soil, soil caves, and different pile-rock contact; Yaokun Li performed theoretical deduction and numerical tests on pile-soil stress ratio of composite foundation in karst area, and provided the approximate range of pile-soil stress ratio. However, stress and strain of piles and soil are non-linearly related and it is impossible for them to reach ultimate state simultaneously. To obtain accurate pile-soil stress ratio through calculation faces many difficulties. Above study work indicates that pile-soil stress ratio of composite foundation in karst area is influenced by existence of karst (soil) caves and requires different design values from those under normal geological conditions, which are mainly represented by:
Due to wavy topography in karst area, two reconnaissance drill holes less than 2m away may expose rock with difference up to several or dozens of meters. E.g. burial depth in the rock in karst area presented in reference [10] is between 187.34.8m with difference of 16.1m.

On one hand, when constructing composite foundation, especially rock-socketed piles, in karst area, the piles may have different length to cause long-short piles; on the other hand, as the pile tips are located at surface of inclined and wavy rock, load-bearing capacity of the pile tips is detrimentally affected which further affects pile-soil stress ratio.

In karst areas, there are usually soft or sand strata directly contacting with rock. When pre-stressed tubular piles passing through the soft or sand strata are constructed, load increase of single pile is insignificant but may dramatically go up with insignificant deformation once it contacts the rock which prejudices stress coordination between pre-stressed tubular piles and soil between piles and causes higher pile-soil stress ratio.

Overlying soil in karst areas is usually not thick between 10~30m and consists of a number of layers with extremely uneven thickness, significantly different properties and hardness, which causes different load bearing capacity of soil foundation and significant difference along depth of same soil layer[10]. Hence, numerous soil layers, various thickness and mechanics difference among soil layers affects load bearing capacity of soil between piles of the composite foundation and therefore affects pile-soil stress ratio.

Soil caves, potential soil caves and subsidence in overlying soil, and flow plasticity of soft soil at the rock surface may cause soil settlement and negative frictional resistance on piles. One extreme situation is that soil between piles at the cave subsidence may only bear very minor load or is even unable to bear any load, which causes foundation failure.

In general, in karst areas, the wavy rock, soft overlying strata or sand strata, load-bearing capacity of the soil and existence of soil caves may all exert significant influence on pile-soil stress ratio of composite foundation.

However, there are minor reports with data of site tests on pile-soil stress ratio of composite foundation in karst areas, which causes parameter design of pile-soil stress ratio lack of support by mass test data when designing composite foundation in karst areas.

In view of this, in this paper, site plate static load tests on composite foundation in karst area are presented, pressure applied on pile top and soil between piles is inspected, pile-soil stress ratio is inspected, and data from tests on pile-soil stress ratio at non-karst areas are compared and analyzed to provide reference for design of pile-soil stress ratio of similar projects in karst areas.

2. Overview of Site Tests
In order to test pile-soil stress ratio of composite foundation in karst areas, site plate static load tests on composite foundation in karst area are introduced in this paper. Pressure cells are installed on pile top and in soil between piles to inspect pressure applied to pile top and soil between piles in order to analyze change trend of pile-soil stress ratio and distribution as increase of load. Object of the site tests is the project of high-rise residential buildings at Liwan District of Guangzhou City.

2.1. Project overview
The project is located on the north of viaduct section of Datansha Subway Line No.5 at Liwan District of Guangzhou City and east of Shuangqiao Middle School. To the east of the site is the Pearl River. The site is located in an area with karst distribution covered by Quaternary system.

Note: The gray part in the Fig.1 indicates distribution of carbonatite. The darker the color means the higher the purity of the carbonatite. Oblique dash part indicates distribution of the overlying layer of Quaternary system. Part of the area is covered with overlying carbonatite of Quaternary system, which is overlay karst distribution.

Geological survey information shows that within the site, there are 51 drill holes exposing limestone or marlstone, and 29 holes exposing karst caves with cave ratio 46.7% which indicates intensely developed karst. Map of geological cross section is shown in Fig. 1. It can be seen from the Fig.1 that
the site is dominated by multi-layered karst caves with single-layered karst caves locally. The caves are generally with no filling and part of them are filled with cohesive soil. The 4.4m moniliform karst cave (ZK30) is the highest and the smallest is 0.16m (ZK16). Geological conditions of the site show apparent geological features of overlay karst area which is presented as minor/medium decomposed rocks with overlying sand. Fig. 1 indicates that roof of some karst caves revealed by drill holes is rather thin.

2.2. Design overview of the composite foundation

The project is to be constructed with three 33-story buildings (100m high). The high-rise buildings are constructed with foundation of rigid – semi-rigid piles (CM pile) + raft foundation. Characteristic value of load-bearing capacity of the composite foundation: 580kPa. Of which, static pre-stress tubular piles (PHC tubular pile) are adopted as rigid piles (C pile), diameter: 500mm, length: 7~20m, pile spacing: 1.5m. High-pressure jet grouting piles (M pile) are adopted as semi-rigid piles, diameter: 500mm, length: 8m, injected down to bottom of the sand layer to form quincuncial configuration together with M piles. Rock and soil layers penetrated by the C piles and M piles and their correlation with karst caves.

Based on formula for calculating load-bearing capacity of composite foundation with CM piles:

\[
f_{spk} = \eta_c \frac{m_{Rc}}{A_{pc}} + \eta_m \frac{m_{Rm}}{A_{pm}} + \eta_s (1 - m_c - m_m) f_{ok}\]

Where, \(f_{spk}\) is the characteristic value of load-bearing capacity of the composite foundation, and \(m_{cm}\) are area replacement ratio of C pile and M pile. Pile spacing for this project is 1.5m with quincuncial form and replacement ratio of 0.1; \(\eta_c\) is service factor of C pile and determined to be 0.95 considering raft foundation adopted; \(\eta_m\) is service factor of M piles and determined to be 0.95; \(\eta_s\) is service factor of soil and between 1.1~1.2 for normal strata and 1.0 for karst area considering development of soil caves within such area.

According to formula (4.3.1) specified in Technical Code for Three Dimensional High-strength Composite Foundation with Rigid – semi-rigid Piles by Guangdong Province, it may be obtained: in composite foundation, load bearing by the pile: 4598kPa, load bearing by M piles: 435kPa, and load bearing by soil between piles: 90kPa, calculated pile-soil stress ratio of C piles: 51.09, pile-soil stress ratio of M piles: 4.83.

2.3. Overview of the tests

In order to inspect pile-soil stress ratio of CM piles, three sets of plate static load tests are performed. Arrangement of the plate tests at base of raft foundation of 5# tower building is shown in Fig.2(a) with test set numbers: YB1, YB2, YB3. Each set of tests includes 2 C piles (pre-stressed tubular piles) and 2 M piles (high-pressure jet grouting piles). Arrangement of the load bearing plates is shown in Fig. 2(b).
2.3.1 Instruments
Counter force device with weight-bearing platform (as shown in Fig. 2) is adopted in the tests which is used to apply counter load force. Considering maximum load value of 5220kN in the project tests, weight on the platform is 6500kN (≥1.2 times of the pres-set maximum load value). Yanhai RS-JYD static load analyzer is used for loading data, measuring and controlling loading which is applied by oil jack. Displacement sensor is used for measuring settlement values.

Pressure sensor: in order to precisely measure pile-soil stress, the pressure sensors are arranged on cushion pads 50mm away from elevation of pile-top to allow soil pressure cells to measure pile-top stress and pressure of soil between piles.

2.3.2 Preparation
Interval between pile completion and start of tests: the pile shaft reaches design requirements for at least 28 days. Meanwhile, following preparation work shall be completed:

The test site shall be leveled off and loose soil be removed. A level surface shall be formed with bearing surface of load plate if the site is located on a slope.

Pressure cells shall be installed on tops of C piles and M piles, as well as in the soil between piles. Before installing the pressure cells, 1~2cm thick fine sand shall be paved on the excavated surface. After pressure cells are installed, the sensors and connections shall be provided with protection means and calibration coefficients of the soil pressure cells shall be well recorded. Arrangement of the soil pressure cells is shown in Table 1 and Fig.3.

| Cell No. | Range Fs | Precision | Position |
|----------|----------|-----------|----------|
| No.1～7  | 1MPa     | 1%Fs      | Soil     |
| No.8～9  | 2MPa     | 1%Fs      | M piles  |
| No.10～11| 5MPa     | 1%Fs      | C piles  |

2.3.3 Loading method and observation of settlement
a. Load is applied in stages with equal increase; the maximum test load is 2 times of the characteristic value of designed load bearing of the composite foundation (1160kPa, 5220kN) and staged load shall be 1/10 of the maximum test load. Style and spacing
b. After applying load of each stage, settlement shall be measured and recorded after 5, 15, 30, 45, and 60 minutes, and then every 30 minutes.

c. Standard for relatively stable settlement of the load bearing plate: when test load is less than load corresponding to the characteristic value, settlement of the plate within one hour is not larger than 0.1mm and is not larger than 0.25mm in every hour when test load is larger than load corresponding to the characteristic value. After settlement rate of the bearing plate meets the standard for relatively stable settlement, load of next stage is applied.

d. The test shall be terminated if one of follows occurs:

e. Vertical deformation is increased dramatically, soil is squeezed out or the plate surroundings upswell significantly; Settlement is increased abruptly (settlement of current stage exceeds 5 times of settlement at previous stage) and a section of sharp drop occurs to load-settlement (Q-s); Under load of a stage, settlement rate within 24h fails to satisfy the standard for relatively stable settlement; Ratio of accumulative settlement to width or diameter of the bearing late is ≥0.06; When load is applied to maximum test load, settlement rate of the bearing plate meets the standard for relatively stable settlement.

f. Releasing load

During the test, whenever one of above conditions for terminating loading occurs, the test will be terminated and the load be released. Similar to loading process, releasing process is also staged with releasing load of each stage equal to 2 times of the loading value. Remaining load of each stage shall be maintained for 30 minutes. Settlement of the bearing plate shall be measured and recorded after 5min, 15min, 30min, and after load is released to zero, as well as 2 hours after load is released to zero.

2.3.4 Data measurement

Deformation of the bearing plate is measured with 4 displacement sensors fixed on datum line beam by magnetic stand.

Load bearing capacity is measured with vibrating wire soil pressure cells. Frequency of the pressure gauge in soil is obtained with dedicated SSC-102 frequency recorder through a wire exposed to the ground and then converted into soil pressure with following formula:

\[
p = K(f_i^2 - f_0^2)
\]

Where: \( p \) is soil pressure kPa; \( K \) is calibration coefficient of the soil pressure gauge; \( f_i \) is test frequency; and \( f_0 \) is initial frequency. Precision of the soil pressure test is 0.1kPa.

3. Test Results

3.1. Analysis on test data

The p-s curve obtained from plate static load tests on composite foundation is shown in Fig.4.

![Fig.4  p-s curve from static load tests](image)

Fig. 4 indicates that when maximum test load applied during the tests reaches 2 times of characteristic value of the designed load bearing, the p-s curve shows no apparent proportional limit and
the composite foundation shows no damage. In view of this, the characteristic value of load-bearing capacity of the composite foundation is determined to be 1/2 of the maximum load applied, which is 530kPa. Load bearing capacity of the composite foundation complies with design requirements. On this basis, characteristic values of load-bearing capacity of C piles, M piles and soil between piles and their corresponding settlement are obtained as shown in Table 2.

| Table 2 Results of static load tests on composite foundation |
|---------------------------------------------------------------|
| No. | Area of the plate/m² | Maximum load/kN | Characteristic value of load-bearing capacity of the composite foundation/kPa | Maximum settlement/mm | Setttement characteristic capacity/mm | Corresponding value of load-bearing capacity |
|-----|----------------------|-----------------|---------------------------------|----------------------|--------------------------------------|--------------------------------------------|
| YB1 | 4.50                 | 5220            | 580                             | 17.44                | 8.19                                 |                                            |
| YB2 | 4.50                 | 5220            | 580                             | 9.36                 | 4.07                                 |                                            |
| YB3 | 4.50                 | 5220            | 580                             | 14.89                | 5.57                                 |                                            |

Results of tests on piles and soil stress indicate that pressure at each point for monitoring piles and soil stress is increased as increase of load and decreased as release of load, which coincides with construction conditions. It essentially reflects change of soil pressure during loading and load releasing process. Change trend of pile-soil stress ratio during loading process at each test point is shown in Fig.5.

![Fig. 5Pile-soil stress ratio during loading ratio](image)

Table 3 Range of pile-soil stress ratio

| Type of pile          | Test point 1     | Test point 2     | Test point 3   |
|-----------------------|------------------|------------------|----------------|
| PHC tubular pile      | 14.5~31.9        | 16.4~30.6        | 24.1~49.0      |
| High-pressure jet grouting pile | 2.6~7.9  | 1.3~6.8    | 1.6~7.0        |

In general, pile-soil stress ratio is related to factors such as material of the pile shaft, diameter and length of piles, replacement rate of area, time, properties of foundation soil, and load level.

Fig.5 and Table 3 show that test results approximately coincide with results of calculation. As load is increased, increase of stress at top of PHC tubular piles is significantly higher than that of high-pressure jet grouting piles and soil between piles. Pile-soil stress ratio of PHC tubular piles is at least >14 while that of cement mixing piles is within 8, which indicates that stress is concentrated at top of PHC tubular piles. Two reasons are related to this: first, rigidity of PHC tubular piles is much higher than that of high-pressure jet grouting piles and soil between piles; second, fine sand layer exists above rock at the test site[10], and during PHC tubular pile driving process, load is increased rather slowly and increased quickly to final load only when it reaches the rock[11]. When final load is reached, comparing to high-pressure jet grouting piles and soil, deformation of pre-stressed tubular piles is smaller and load-bearing capacity is higher. High-pressure jet grouting piles have smaller pile-soil stress ratio because strength of the pile shaft is much lower than pre-stressed tubular piles, and they are mainly used for soil reinforcement to improve load bearing capacity of soil between piles and provide lateral bracing for pre-stress tubular piles.

During the tests, pile-soil stress ratio is increased as increase of load, while according to references[12], pile-soil stress ratio of deformable piles decreases as increase of load. The main difference is that during loading process, rigid piles take more and more load due to their significant rigidity, strong self-stabilization and deformation discordance between piles and soil.

Test results indicate that pile-soil stress ratio is not a fixed value and pile-soil stress ratio of rigid piles is increased as increase of load. Given this fact, calculation of load-bearing capacity of composite...
foundation shall be on the basis of load, and different pile-soil stress ratio shall be used for calculation for composite foundation.

3.2. Load bearing capacity of C piles
As per DBJ/T 15-136-2018 Technical Code for Building Foundation in Karst Area, in addition to plate tests for verifying load-bearing capacity of composite foundation, the composite foundation in karst area shall also be tested to verify load-bearing capacity of single pile with diameter of 500mm and shaft of pre-stressed tubular pile (PHC tubular pile). Arrangement of test points for verifying load-bearing capacity of single pile is similar to those for plate tests on rock and soil of composite foundation nearby.

Fig.6 Results of tests on load-bearing capacity of single pile
According to results, under current geological conditions, load-bearing capacity of single pile is relatively high up to 3600kN (intensity of pressure of load-bearing capacity: 18335kPa). Load bearing capacity of C piles is not exhausted with considerable redundancy. It is because design value of load-bearing capacity of single C pile (which is 950kN, and intensity of pressure: 4838kPa) is rather low in order to relieve effect of wavy rock on pile pressing in karst area. Design value of load-bearing capacity of single M pile is 80kN. This indicates that C piles are provided with higher safety margin, which complies with the design requirements for reducing load-bearing capacity of single C pile.

Reducing design value of load-bearing capacity of C piles may reduce pile-soil stress ratio, therefore to allow soil between piles to better bear load. On this basis, composite foundation with rigid piles may be effectively improved by improving pile-soil accordance through increasing thickness of foundation bed and rigidity of raft.

3.3. Comparison and analysis on pile-soil stress ratio in karst areas and non-karst areas
Comparison between pile-soil stress ratio of CM piles obtained from tests in karst area presented in this paper and pile-soil stress ratio in non-karst area is shown in Table 4.

| Type of pile | Karst area | Non-karst area* |
|-------------|------------|-----------------|
|             | YB1        | YB2             | YB3             | 1# test point | 2# test point |
| C pile      | 14.5~31.9  | 16.4~30.6       | 24.1~49.0       | 10.77~25.62  | 10.81~47.16  |
| M pile      | 2.6~7.9    | 1.3~6.8         | 1.6~7.0         | 1.51~1.88    | 2.13~6.22    |

*Note: Data of pile-soil stress ratio in non-karst areas is from reference [6].

According to comparison of data from karst area and non-karst areas as stated in Table 4, considering geological features of the site and overlying minor/medium-decomposed rock (including karst caves), pile-soil stress ratio of pile foundation in karst areas is higher. Load of pre-stressed tubular piles is abruptly increased when piles pass through sand and contact with the minor/medium decomposed rock. After completion of construction, rigidity of pre-stressed tubular piles is even higher due to high strength of the rock that the piles directly contact with. Besides, due to lowload-bearing capacity of soil between piles in karst areas, especially overlay karst areas, rigid piles bear higher load.
In addition, due to soil reinforcement by the high-pressure jet grouting piles, vertical bearing capacity of PHC tubular piles is reinforced, which causes higher load on PHC tubular piles under the raft and bed, sometimes even higher than designed load bearing. Considering factors such as significantly wavy rock and soil caves in karst areas, if designed load bearing capacity of PHC tubular piles is too high, probability of broken piles may be increased. Hence, designed load bearing capacity of PHC tubular piles of composite foundation shall be moderate and designed with safety margin.

According to construction records, among 1582 C piles for this project, 15 piles were broken during pressing process, probability of broken piles: 1%. Observed settlement results during construction of upper structure are shown in Fig 7.

![Fig.7 Curve from observation on foundation settlement](image)

According to curve from observation on foundation settlement, maximum foundation settlement is less than 7mm. Load bearing capacity in this project in karst area complies with requirements with favorable deformation control. This indicates that lowering design value of load-bearing capacity of C piles may allow soil between piles to better bear load and foundation settlement to be advantageously controlled.

4. Conclusion
This paper analyzes and discusses pile-soil stress ratio of composite foundation in karst areas by combining geological features of project site and site tests on pile-soil stress ratio of composite foundation in karst area, with following conclusions drawn:

In karst area, pile-soil stress ratio of the composite foundation is significantly influenced by wavy rock, weak or sand layers on top, load-bearing capacity of soil and existence of soil caves.

Pile-soil stress ratio of both pre-stressed tubular piles and high-pressure jet grouting piles increase an increase of load. Relative to high-pressure jet grouting piles, pre-stressed tubular piles have higher pile-soil stress ratio. The main reasons are that strength of high-pressure jet grouting piles is lower and purpose of them is to provide soil reinforcement in composite foundation to improve load bearing capacity of soil between piles and lateral bracing for pre-stressed tubular piles.

Higher pile-soil stress ratio of pile foundation in karst areas is mainly related to abrupt change of bearing stratum at pile top and lower load bearing capacity of soil between piles.

Designed load-bearing capacity of PHC tubular piles of composite foundation in karst areas shall be moderate and provided with safety margin.

Pile-soil stress ratio is not a fixed value and shall be increased as increase of load, and design with various pile-soil stress ratio shall be adopted. Pile-soil stress ratio may be lowered by decreasing designed load bearing capacity of C piles therefore to allow soil between piles to provide better load bearing.
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