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A Review on Interaction Effect of Pile under Vertical Compression

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Abstract: *The present theories and methods for design of the vertically loaded pile foundations generally provides the overall values of bearing capacity and settlement characteristics for the pile groups. In every design criteria for vertically loaded pile system, there are some factors affecting on major scale such as bearing capacity, Settlement, skin friction etc. This review paper shows that the consequences of vertical load on pile foundation done by various researchers and from their results help in predicting the load–settlement behavior of vertically loaded pile and pile groups. It is observed, when the group efficiency is close to unity, the variation between expected and observed behavior of piles proves to be a positive approach towards the ultimate load. Hence, various researchers have tested and published their laboratory and theoretical values on load-settlement characteristics under the influence of vertical load on pile foundation. This paper illustrates about comparison of various test results and their research gap on consequences of vertically loaded pile foundation.*

Keywords: *Pile bearing capacity, Settlement, Interaction effect Model test, Ultimate load;*

I. Introduction

In every pile foundation group of piles are common approach to achieve higher strength and this pile groups are placed apart that they do not admit any effect to each other. The previous research shows that the total ultimate load produced by pile group is not simply the multiplication of the total ultimate load of single pile. In clay load may be less and in sand it may be greater than this value. Similarly the settlement of a pile group subjected to average load per pile it may be very different from the single pile settlement under the given average load.[1]

Many constructions supported by pile system under vertical loading, reason behind of that to decrease deformation at certain limit to an acceptable level. This can be done by pile-soil-pile interaction effect where piles are arranged in groups. In every scenario of the pile load-settlement, it shows that pile spacing in group pile system plays major role. This review paper reflects some experimental work done on pile foundation by different researchers and comparison of their results.[3]

II. Review of Previous Work Done

A. Load test on Pile group in Sand- Vesic (1967)[15]

Vesic (1967)[15] presented the experimental vertical load test results of a prototype model pile group which consists of 9 circular aluminium tube pile connected by rigid cap with pile-soil-cap interaction effect. The pile had a thickness of 1.27mm, an external diameter of 101.6mm and length of 1524mm. This prototype model scale test performed at Georgia Technical University (Atlanta, Ga.) in a medium dense sand formation. Nine circular piles are arranged in a 3x3 configuration with pile spacing of 2d. The pile group efficiency was governed by O'Neill (1981)[10] at the failure load of 1.21. He attributed such a value to the confinement and densification effects produced by pile installation. The soil properties are such as the relative density, Dr = 69% and unit weight, γ_s = 14.9 kN/m3, he provides the use of the following expression for estimation of the soil modulus, E_L, at the pile tips:

$$E_L = 18.75\gamma_s L N_q(1 + D^2) \quad (1)$$

Where, Nq is a bearing capacity factor(deduced from the tip capacity of the single pile).

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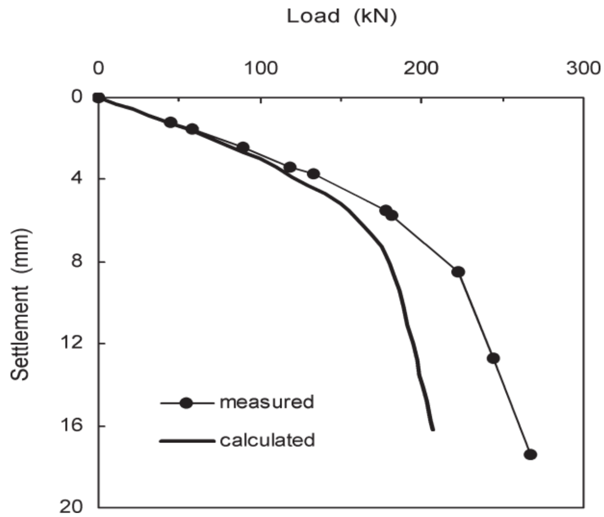

Fig. 1: Comparison of measured and predicted load-settlement behaviour for pile group-vesic(1967)

The consequences of measured and calculated pile load settlement compared in (Fig. 1). As per failing geotechnical theory, in this study the subsoil has been Gibson-type soil which having small stress-strain modulus linearly increase with the increase in depth calculated using eq. (1) at pile tip where the value of N_q is 40 (O'Neill 1981)[10].

As per comparison results (Fig. 1), measured and calculated values of load-settlement is rather satisfactory, except at higher load value where the measured settlements are substantially lower than calculated settlement and predicted ultimate load of pile group is about of 20%.

B. Bearing capacity of pile foundation- Koizumi and Ito (1967)[8]

The field test and bearing capacity regard to pile foundation presented by Koizumi and Ito (1967)[8]. The authors study concern a 3x3 pile group of steel piles which driven into a highly sensitive clay medium resting on sand-gravel layer up to a depth of 14m from the top surface of the ground.

The dimensions of the piles were 3.2mm thickness, 300 mm diameter tubular pipes and length of 5.5 m. These piles were placed at spacing of 3d and connected by a pile cap which contact with the soil. Tests were conducted on isolated single pile and pile group. The pile group efficiency (i.e., the ratio of the pile group failure load to the isolated single pile failure load multiplied by the number of piles in the group) was taken about 0.67(O'Neill 1981)[10]. The undrained cohesive strength was about 25 kPa at the pile head which deducted from the laboratory and in-situ tests increases up to 40 kPa at the pile tip. Elastic modulus linearly increases with an increase in depth. The value of elastic modulus found by eq. (2)

$$E_s = \beta S_u \tag{2}$$

where β is an empirical factor.

Assuming for soil model, a value of $\beta = 520$ has been taken from the back-analysis of the small-strain stiffness of isolated single pile deducted from the loading tests. A comparison of measured and theoretical load-settlement curves for the pile group as shown in (Fig. 2). Due to the lower pile group efficiency, the discrepancies between measured and predicted behavior at very high load are more pronounced.

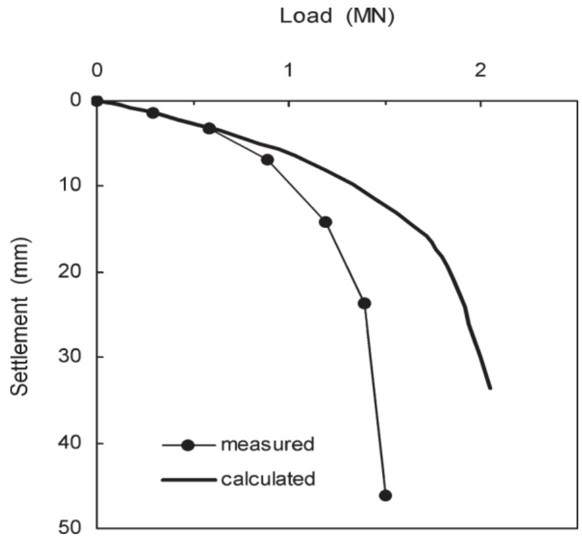

Fig. 2: Comparison of measured and predicted load-settlement behaviour for pile group-koizumi and ito (1967)

C. Load Test on two full-scale pile groups in clay-O'Neill(1981)[10]

The full-scale pile group load test described by O'Neill (1981)[10], conducted by American Railway Engineering Association. This test had performed on a 9 pile with 3x3 configurations in a soft to medium stiff clay. A schematic diagram of pile system is shown in (Fig. 3).

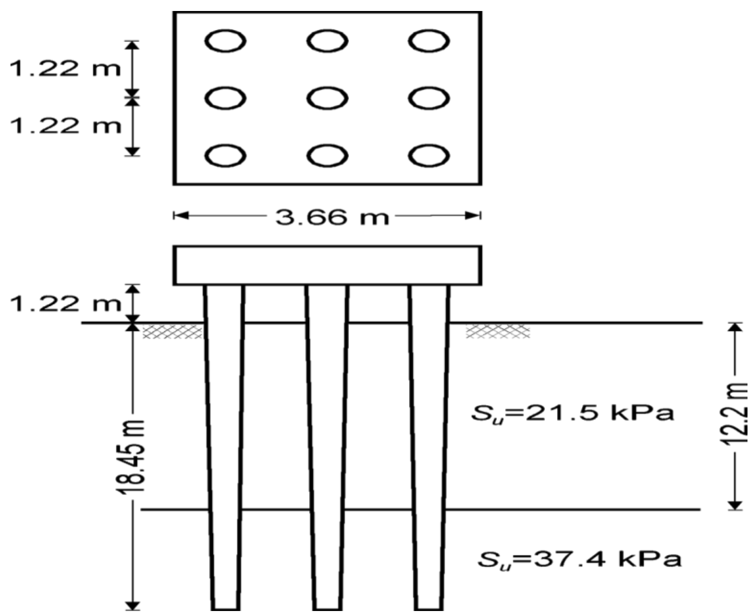

Fig. 3: Schematic diagram of pile system

The piles were tapered steel piles having diameter of 419mm at the head and 203mm at the tip of pile. The embedded length of pile is 18.45 m and wall thickness is 4.6mm. Piles were driven at a spacing of 1.22m. Here, pile cap was not contact with the soil surface. The pile group efficiency with respect to the allowable failure load was about 0.75.The subsoil strata consisting of two-layer soil system of saturated clay.

The depth of upper layer is 12.2m. The average values of the undrained shear strength, Su, are 21.5 and 37.4 kPa for upper and lower layers respectively. The soil modulus values were not provided by the author however, the load-settlement curve presented of a reference pile. For analyzing the response of pile group, O'Neill is assumed that soil modulus is directly proportional to the undrained shear strength as per eq. (2).

The author also performed some theoretical analyses which considered different values of β and taking the best values from observed and predicted when β = 2000. In these analyses, evaluation of this factor was done by matching the measured small-strain stiffness at the reference pile head. The value of Poisson's ratio is 0.5 and β = 3000.

The experimental Load-settlement results were compared in (Fig. 4). The elastic shortening of the piles has been also taken into the analyses. As per results the author noted that, present solution allow a good prediction of the load-settlement even in vicinity of actual results of load-settlements. However, the significant overestimated with respect to the measured by a factor of about 25%., which shows the group efficiency value.

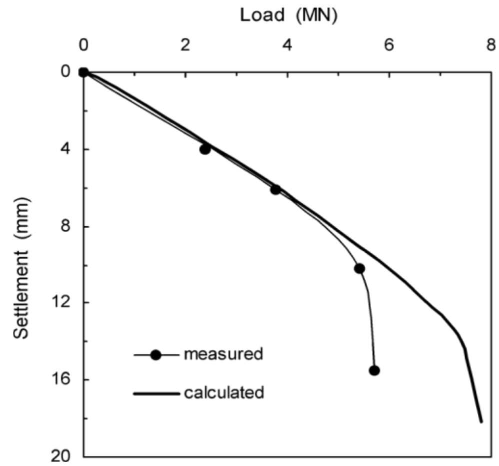

Fig. 4: Comparison of measured and predicted load-settlement behaviour for pile group- o'neill (1981)

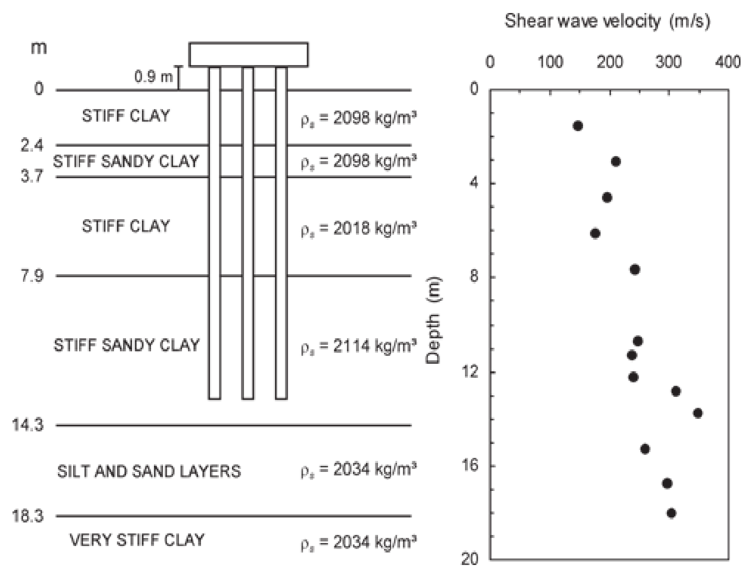

Fig. 5: Layout of pile group, soil profile and soil properties

D. Load Test on full-scale pile groups in layered cohesive soil-O'Neill et al.(1982)[12]

The experimental results of the vertical loading on pile groups presented by o'neill et al. (1982)[12]. These tests were consists of two single piles and pile group. Pile group were arranged in a 3x3 configurations and having 3d pile spacing. Piles were made up of steel and having closed-ended pipes. The dimensions of pile system having a wall thickness of 9.3mm, external diameter of 273mm and they were driven to a depth up to 13.1m. The rigid concrete pile cap placed at 0.9m of the ground on the top of the pile group. A typical layout of pile foundation and average load-settlement curve for two single piles are shown in (fig. 6 and 7), respectively.

As shown in (fig. 7), a pile load-settlement behavior experienced a peak governed by a load softening throw continuous deformation. In the test, corner piles were removed from the cap and other remaining piles were tested in compression in five pile subgroup after testing the pile group. After that, a subgroup of four piles was also tested after removing the centre pile from the cap. As per (O'Neill et al. 1982)[12], the efficiency of all these groups was about 1.

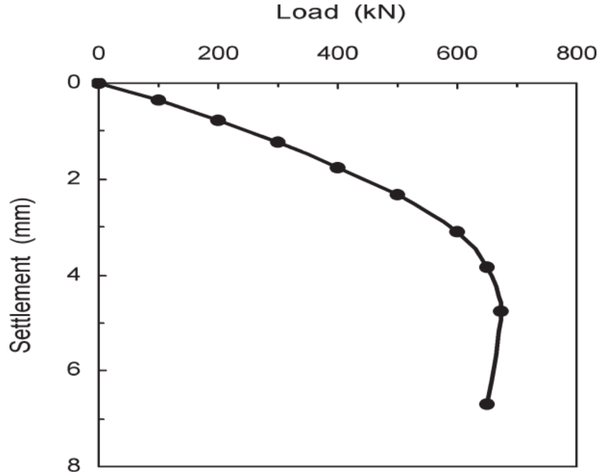

Fig. 6: Average load-settlement curve of the isolated piles- o'neill et al. (1982)

These tests were conducted in the campus of the University of Houston (Houston, Tex.). The cohesive soil profile which was used in the tests, whose over consolidation ratio ranges from about 11 and 4 at a depth of 2m and 14m respectively. The soil properties as shown in (Fig. 5) derived from cross-hole tests (Mahar and O'Neill 1981)[10]. From (Fig. 5), it was proved that two primary strata exist in upper 14.3m; 7.9m of a stiff clay and below that stiff sandy clay. Sand and silt layers were resting on very stiff clay below the depth of 14.3m. Poisson's ratio was taken about to 0.5 (O'Neill (1981)[10].

The soil profile and its properties reported in (Fig. 5) have been taken in conjunction with the f_n values as per test results shown in (Fig. 6). The comparison of the calculated and measured load-settlement behaviour is shown in (Fig. 7). As can be seen, there is very good agreement between the results, on behalf of that some discrepancies between calculated and measured behavior occurs at very high load levels.

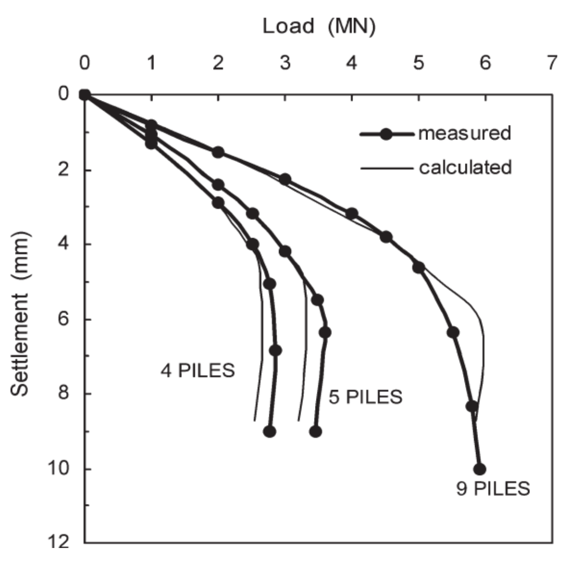

Fig. 7: Comparison of the calculated and predicted load-settlement behavior for pile groups- o'neill et al. (1982)

The Average loads calculated and measured at the corner, edge and centre of piles of the 3x3 pile group for two values of the total load, i.e., 2581 and 5770 kN (Table 1). These two reference loads should be representative for the pile performance under working and failure conditions respectively. Higher load measured at the central pile when the load was about 5770 kN, that was not predicted by present solution of O'Neill at al.(1982)[12] .

Table No. 1: Comparison of calculated and measured load at centre, edge and corner for two values of the total laods- O'Neill et al. (1982).

Group Load (kN)	Pile	Load (kN)	
		Measured	Calculated
2581	Corner	295	293
	Edge	286	284
	Centre	272	274
5770	Corner	644	652
	Edge	623	636
	Centre	704	617

E. Test on Axially loaded five pile group and single pile in sand- Briaud et al. (1989)[2]

A single isolated pile and a five group piles were tested and analyzed by Briaud et al. (1989)[2]. The tabular sheet piles having a diameter of 273mm and thickness of 9.3mm were driven to a depth of 9.15m below the ground surface and connected by a rigid cap (Fig. 8).

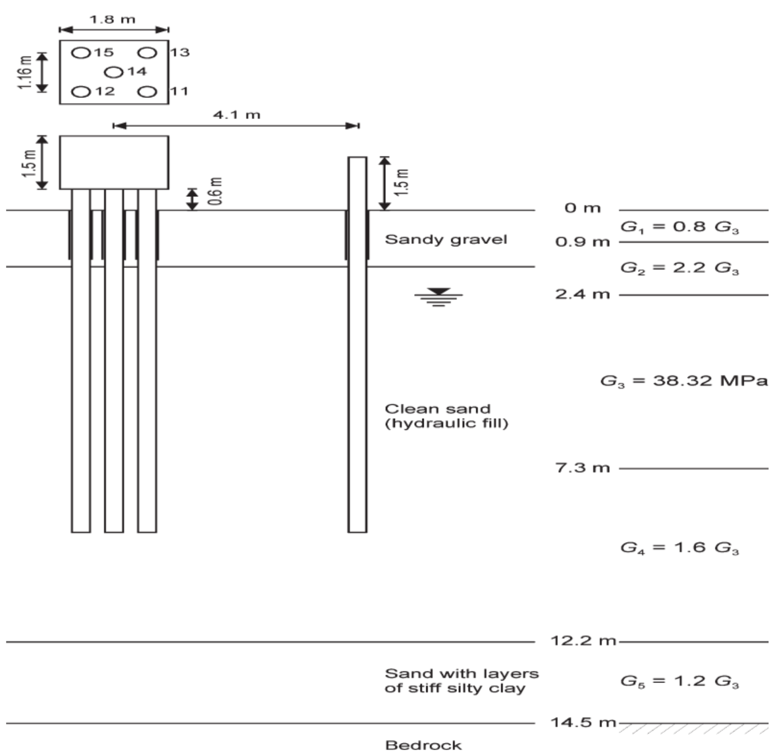

Fig. 8. Layout of the pile group and soil profile- briaud et al. (1989)

The pile group efficiency was taken about 0.99. The test was conducted in San Francisco, where subsoil layer having different soil strata. A first layer of sandy gravel 1.37m thick and it was overlies a hydraulic fill made of clean sand up to a depth of 12.2m; stiff silty clay layer interconnected to the bedrock about 14.5m. The water table was 2.4m below the ground surface. The values of small-strain shear modulus (G) of each soil layer founded by the field investigation carried out by the authors.

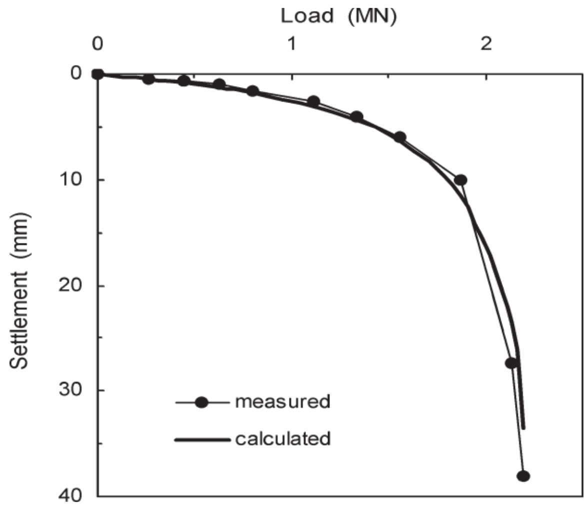

Fig. 9. Comparison of the calculated and predicted load-settlement behavior for pile groups- briaud et al. (1989)

The shear modulus ratio of different layers was taken from cone penetration test profiles. Poisson's ratio assumed to 0.15 throw out the depth. The modeled soil profile was similar to the model considered by Mandolini and Viggiani (1997)[9]. The comparison of calculated and measured pile-load settlement is reflected in (Fig. 9), where a very small agreement between those values observed for failure load. Even after that, (Fig. 10 and 11) show the observed load as a function of total load acting on the pile cap described by the central and corner piles, respectively. As can be seen, this proposed method provides satisfactory predictions on the measured load distribution among the piles.

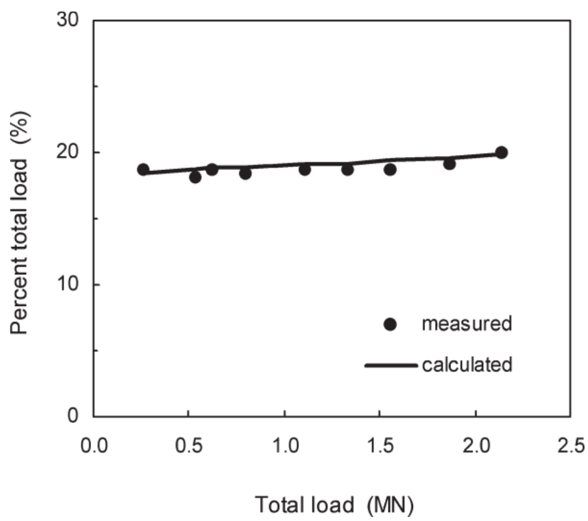

Fig. 10. Comparison of the calculated and predicted loads carried by the central piles -briaud et al. (1989)

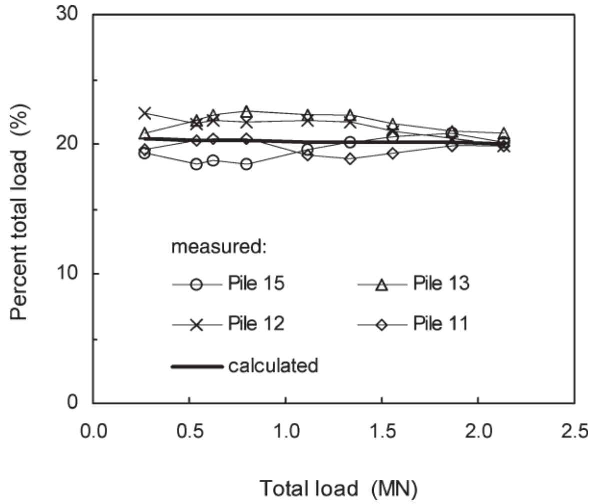

Fig. 11. Comparison of the calculated and predicted loads carried by the corner piles **-** briaud et al. (1989)

F. Centrifuge tests on a large pile group in clay- Conte et al. (2003)[4]

Conte et al. (2003)[4] performed various centrifuge tests on piled rafts and their individual components i.e. single pile, single raft and group of pile. These all tests performed on a prototype scale model. The aim of the study was to find the response of single pile and 7x7 group piles have been considered by the author. The prototype scale pile having a diameter of 0.63m, length of single pile is 17.5m and length of piles in a group is 18.5m. The piles were placed at a spacing of 4d which connected by stiff free standing raft. The load-settlement behavior of a single pile is shown in (Fig. 12) with the fitting curve to evaluate f_n as a function of the load level.

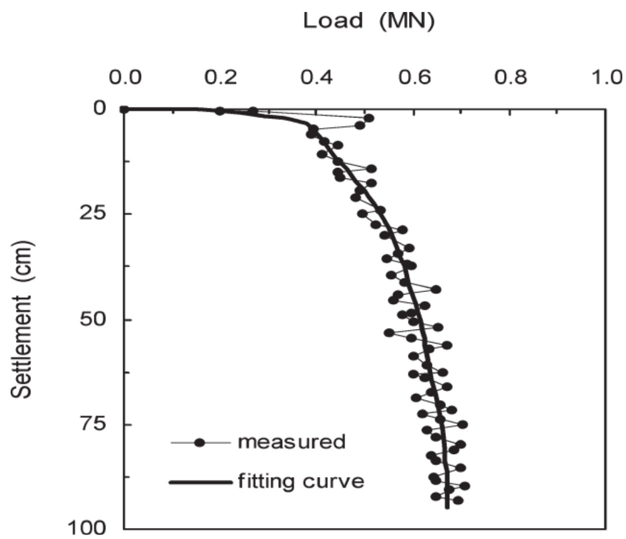

Fig. 12. Load-settlement response of single pile with indication of fitting curve - conte et al.(2003)

The single pile's small-strain stiffness was 39.4 MN/m, and pile group efficiency ranged from 0.82 to 0.90 depending on the settlement. The soil was kaolin clay and its profile can be presented by the following expressions (Conte et al. 2003)[4]

$$S_u = 18 \text{ kPa} \quad \text{for } 0 \le z \le 13.8 \text{ m} \tag{3}$$

$$S_u = 1.304z \quad \text{for } z > 13.8 \text{ m} \tag{4}$$

Where z denotes the depth(m), and S_u is the un-drained shear strength of the soil(kPa), The soil modulus was expressed by (eq. 2). When β = 210, the measured initial stiffness of the single pile was 39.4 MN/m.

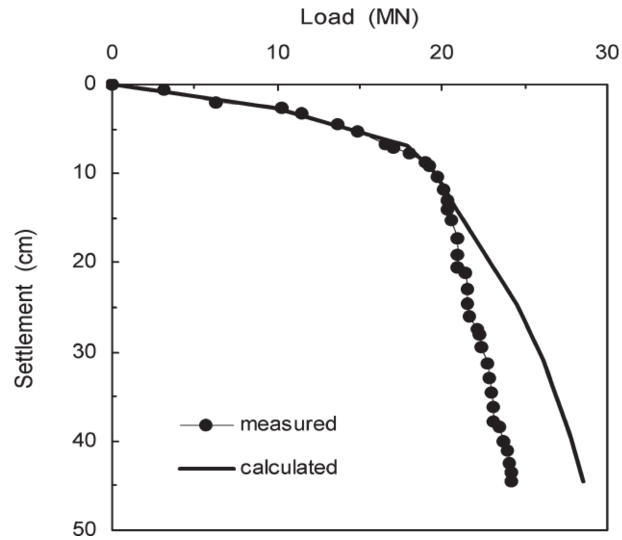

Fig. 13. Comparison of the calculated and measured load-settlement behavior for the pile group - conte et al. (2003)

The measured and calculated load-settlement curves for the group piles are compared in (Fig. 13), which reflects again there was very close agreement between the measured and calculated results except at very high load level. Here the theoretical curve shows significant underestimated settlement and overestimated the load about 15%.

III. Results

From the various experimental results, it can be noticed that the present method for predicting the load-settlement behavior of vertically loaded pile groups generally capacble. As per this study we proved that, at very high level of loading, the calculated results may vary significantly from those measured when the pile group efficiency with respect to the failure load is differ from the unity[12]. These variations should be major attributed to some technological factors which was connected with the soil profile. Generally, the major factor which affecting on higher scale such as the ultmate bearing capacity responced for deformation of the pile foundation. Further that, when the pile group efficiency is closed to unity, the small agreement between calculated and measured behavior states to be very satisfactory up to the ultimate load.

IV. Conclusion

From the various experimental results, it can be noticed that the present method for predicting the load-settlement behavior of vertically loaded pile groups generally capacble. As per this study we proved that, at very high level of loading, the calculated results may vary significantly from those measured when the pile group efficiency with respect to the failure load is differ from the unity[12].

These variations should be major attributed to some technological factors which was connected with the soil profile. Probably, the small-strain soil shear modulus profile described by in-situ geotechnical measurements and load-settlement behavior of the single pile gradually deducted from the loading test.

The comparison of different experimental measurements from various well-documented vertical or axial loading tests on isolated single pile and pile group, provide satisfactory agreement between measured and calculated load-settlement behavior, even ultimate load approach proved that the pile group efficiency with respect to the failure load is close to unity. From this study, the spacing of pile in pile group also conducts an agreement with the load bearing capacity of pile group[12]. With increase in pile spacing the load-settlement behavior of pile foundation also increase.

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