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Problems of evaluating the multi-level bidirectional test results of a large diameter bored pile in Mekong delta, Vietnam

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ABSTRACT

This paper addresses the problems of evaluating the multi-level bidirectional test results on the large diameter long bored pile in Mekong Delta area, Vietnam. The soil profile consisted of silty sand deposited on clayey silt and underlain by dense silty sand. The shaft resistances measured below the lower cells were about 4 to 8 times greater than between the two cell levels in the similar soil condition. The analysis finds that the shaft resistances measured between the lower cell and pile toe has been strongly governed by the pile toe stiffness, whereas the shaft resistances between the two cell levels have been significantly influenced by multi-stage load tests.

Keywords: bidirectional test, bored pile, pile toe stiffness, shaft resistance, pile axial stiffness

1 INTRODUCTION

The multi-level bidirectional tests are commonly applied into the large diameter long bored piles. For two-level bidirectional tests, the loading is often started by the lower cells and followed by the upper cells.

Two problems often observed in the pile test results are: 1) the shaft resistance below the lower cell is significantly greater that above the lower cells; 2) the shaft resistance between the two cell levels measured by the loading of the upper cell is dramatically influenced by the loading of the lower cells.

This paper will examine such problems on the two-level bidirectional load test result of the bored pile in 2.5-m diameter and 84-m length at the Cao Lanh cable-stayed bridge in Mekong delta of Vietnam.

2 SOIL PROFILE

The soil profile near the test pile consists of surficial layers of silty sand and clayey silt to 9 m and 17 m depths below elevation of river bed, respectively, deposited on medium stiff clay to 26 m depth, followed by very stiff clay to 40 m depth and underlain by dense to very dense silty sand. Figure 1 shows the distribution of water content, consistency limits, grain-size distribution, and SPT N-indices determined from the borehole records. The average saturated density and water content of the clay were about 1,800 kg/m^3 and 34 %, respectively. The average density of the sand was about 1,800 kg/m^3. From 13 m through 21 m depths (loose silty sand), the average SPT N-indices was about 3 blows/0.3 m. Following 21 m depth to 59 m depth, the SPT N-indices increased from about 3 blows/0.3 m to about 16 blows/0.3m, indicating soft to very stiff clayey silt. Below 59 m depth, the N-indices showed the conditions to be very dense.

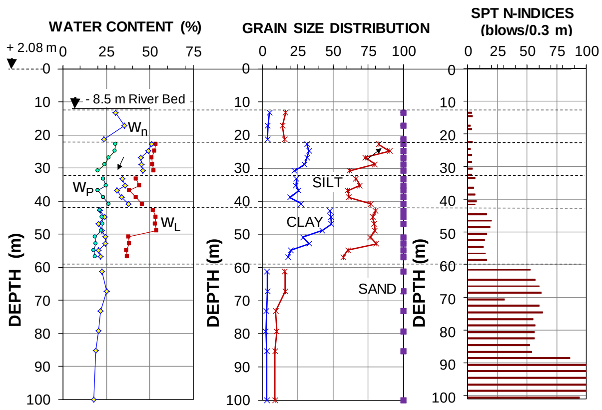

Figure 1. Water content, soil type proportions, and N-indices.

3 CONSTRUCTION OF TEST PILE

The test pile in 2.5-m diameter was constructed using bucket drill technique with casing advanced ahead of the hole to about 84 m depth below the river water level on October 19, 2014 (Figure 2). The drilling of test pile was done under bentonite slurry. Before concreting, the shaft was cleaned and a reinforcing cage with the Cell assembly attached at 1.5 m and 13.5 m above the cage end was lowered into hole.

Two pairs of diametrically opposed vibrating wire strain-gages were attached on cross-section area of test pile at a level below the lower cell, two levels between two cell levels, and two levels above the upper cell, as shown in Figure 2. 35 days after completed concrete, the pile test was performed. The 28-day concrete strength of the test pile was about 46.8 MPa.

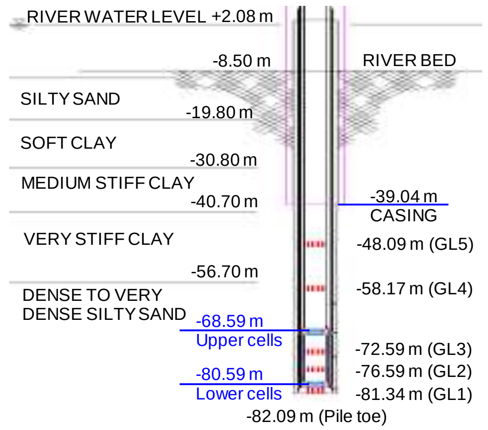

Figure 2. Details of strain gages installed in pile.

4 LOADING SCHEDULE

The pile test was perfomed in three loading stages (Figure 3). In the Stage 1, the loading was performed by increasing pressure in the lower cells in 16 equal increments to reach a maximum bi-directional load of 12.4 MN. Then, the unloading was done in the eight decrements. Each of load increments and decrements was held during 15 and 10 minutes, respectively.

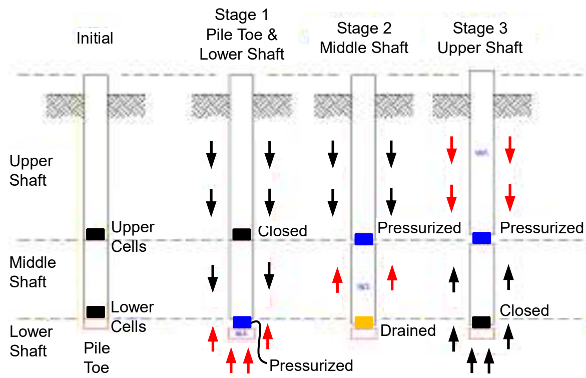

Figure 3. Loading procedure of two-level bidirectional test.

In the loading Stage 2, the loading of the upper cells was performed in 12 equal increments to a load of 6.2 MN. For 12 increments, the lower cells were left free to drain (no load transfer through the cells to end bearing). After mobilizing fully the shaft resistances between the two levels of cells, the loading Stage 3 was started. The lower cells were closed off and the loading of the upper cells continued to assess the shaft resistances above the upper cells by using the skin friction bellow and the end bearing as reaction. The additional loading was loaded in 10 equal increments to a maximum load of 14.5MN and then the unloading was done in six decrements. Each of the load increments and decrements was held during 15 minutes. The last load decrement was held for 30 minutes.

5 TEST RESULTS AND ANALYSIS

5.1 Load versus movement

Figure 4 provides the load-movement curves measured at the lower cells and at the pile toe in the loading Stage 1. Loads measured are not adjusted for pile weight and water pressure at the cell levels. The maximum test load was about 12.4 MN. The maximum downward and upward movements at the lower cells were about 11.8 mm and 0.7 mm, respectively. The maximum movements at pile toe were about 9.6 mm and the maximum shortening of pile shaft was about 2.2 mm. The measurements showed the soil below pile toe is relatively soft, not reflecting the actual condition of very dense sand.

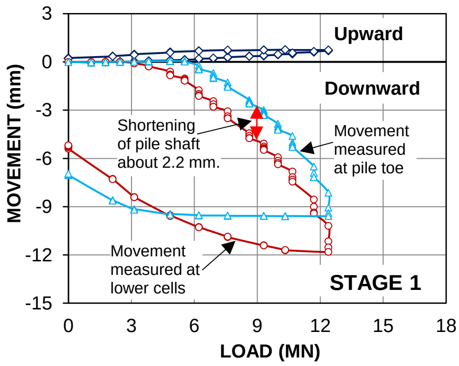

Figure 4. Load-movement curves - Loading Stage 1

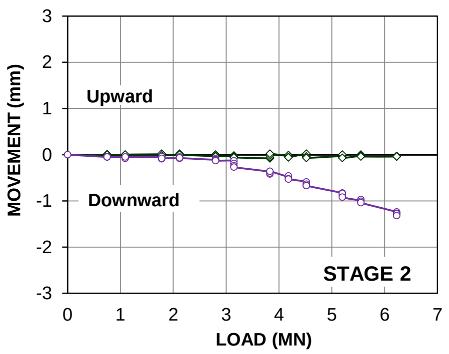

Figure 5. Load-movement curves - Loading Stage 2

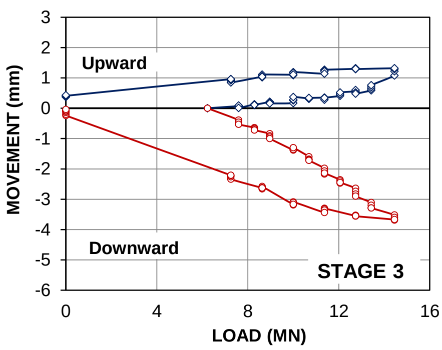

Figure 6. Load-movement curves - Loading Stage 3

Figure 5 presents the load-movement curves measured in the loading stage 2. The maximum upward and downward movements were about 1.3 mm and 0.0

mm at the maximum load of 6.2 MN, respectively. It should be noted that the pile segment between two cell levels is 12 m, which is corresponding to a nominal shaft area of 94.2 m^2. If ignoring the influences of the loading Stage 1, the unit shaft resistance between two cell levels, computed basing on the test results in this loading Stage 2, is about 66 kPa.

Figure 6 shows the load-movement curves measured in the loading stage 3. The maximum net test load in the Stage 3 was about 8.3 MN (excluding the maximum test load in Stage 2). The maximum upward and downward movements were about 1.3 mm and 4.7 mm, respectively. For the maximum upward movement of 1.3 mm, it is difficult to mobilize fully the upper shaft resistances and thus is not discussed in details.

5.2 Load versus strain and the shaft resistances

For the subject case, the measurements of the strain gages GL1 and GL2 in the loading Stage 1 are very important to evaluate the shaft resistance in silty sand layer because the loading to 12.4 MN was enough to mobilize fully the shaft resistance from GL1 through GL2 and the shaft resistance was not influenced by multi-stage loading. Therefore, the measurements of these two gage levels are considered in details. The strains measured by the other strain gage levels were relatively small, not meaning for analysis and also reported by Phung et al. (2016).

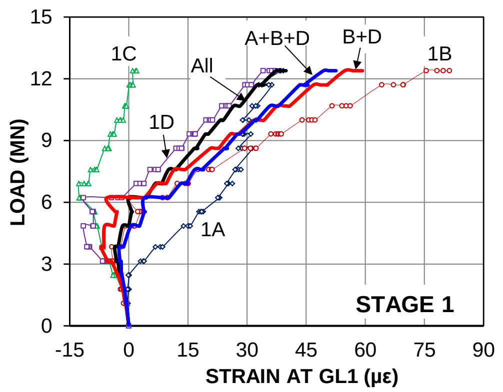

Figure 7. Load versus strain measured at level GL1

Figure 7 shows the load versus strain of GL1 measured in the test Stage 1 and the average strains computed from 2 strain gages (GL1B and GL1D), 3 strain gages (GL1A, GL1B and GL1D) and 4 strain gages. It is noted that most of the measurements of GL1C were less than zero and the strains measured from GL1A did not increase after load increment of about 8.5MN. Thus, taking average strains of the GL1A and GL1C is ignored for analysis. As can be seen clearly from diagrams on Figure 7, the average strains of the strain gages are significantly different.

To select the reasonable strains for analysis, it is necessary to consider the measurements of GL2 (Figure 8). The average strains of each diametrically opposed pair or all strain gages of GL2 are relatively similar. Thus, it is reasonable to take the average strains of 4 strain gages for analysis and it is a good resource to refer for evaluating strains measured at GL1. The pile stiffness estimated basing on the slope of this load-strain curve is about 186 GN, which is corresponding to an E-modulus value of 40 GPa on the nominal cross-section area of pile.

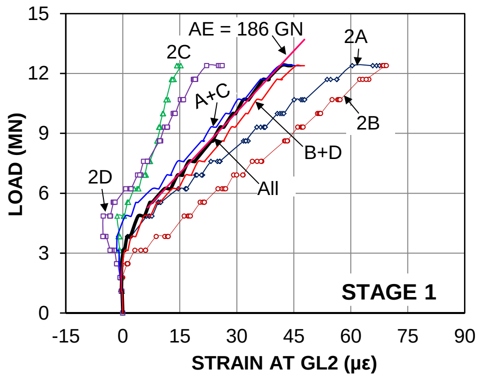

Figure 8. Load versus strain measured at level GL2.

Figure 9 show a comparison of the three average strains of GL1 and the average strain of 4 strain gages of GL2. As indicated in Figure 3, distance from the lower cells to GL1 and GL2 are 0.75 and 4.00 m, which are corresponding to the nominal shaft areas of 5.9 and 31.4 m^2, respectively. Moreover, these two strain gage levels were installed in a similar soil condition. Thus, the strains measured at GL1 will be greater than at GL2 under the similar load conditions of the lower cells. However, the average strain of 4 strain gages of GL1 is smaller than of GL2 (Figure 9) and it is apparent that the average strain of 4 strain gages of GL1 is not reasonable.

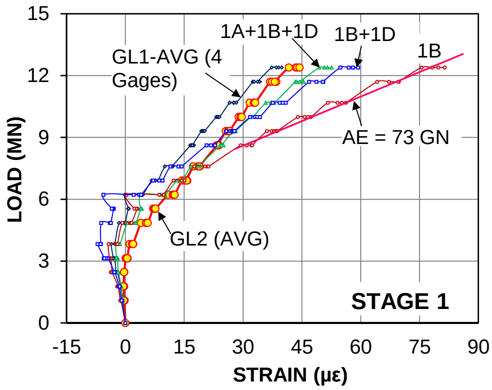

Figure 9. Comparison of the three average strains of GL1 and the average strain of four strain gages of GL2.

The average strains of 2 strain gages (GL1B and GL1D) and 3 strain gages (GL1A, GL1B and GL1D) are only more reasonable than of 4 strain gages after load increment of 9 MN. However, they are not reasonable if considering the nominal shaft area ratio of these two pile segments (the nominal shaft area of lower cells-GL2 is 5.3 times greater than lower cells-GL1). In this case, it seems that the strain measurements of Gage 1B are the most reasonable to consider the shaft resistance below the lower cells. The pile stiffness estimated basing on the slope of this load-strain curve is about 73 GN, which is corresponding to an E-modulus value of 15 GPa on the nominal cross-section area of pile.

Figures 10 and 11 show the unit shaft resistances versus movements computed basing on the pile stiffnesses of 73 GN and 186 GN. As can be seen from diagrams on Figure 10 and 11, the unit shaft resistance from lower cells to GL2 is 4.0 through 8.0 times less than from lower cells to GL1 for both values of the applied pile stiffness. This is not reasonable due to both these gages installed in same soil condition.

To estimate a reasonable value of shaft resistance, shaft resistance of the bored pile in sand was estimated basing on method of Meyerhof (1976) and Decourt (1989) and the standard penetration test results.

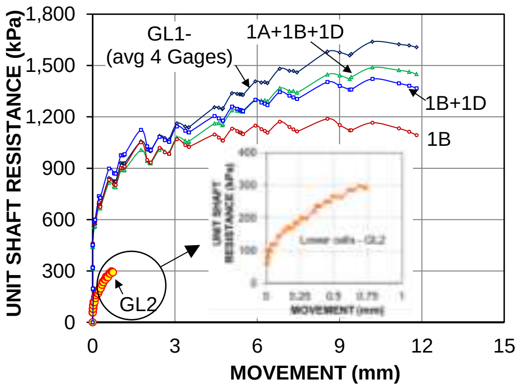

Figure 10. Unit shaft resistances versus movements computed basing on the pile stiffness of 73 GN.

The average SPT N-indices of this pile segment (Figure 1) is about 55 blows/0.3 m and the calculations show the unit shaft resistances of about 55 and 98 kPa for method of Meyerhof (1976) and Decourt (1989), respectively. It has become clearly that the unit shaft resistances estimated from SPT N-indices are significantly smaller that obtained from the GL1.

If comparing the unit shaft resistance measured from the lower cell to GL1 in the loading stage 1 (from 1,200 to 1,500 kPa as shown on Figure 10) with that measured from the upper cell to the lower cell in the loading Stage 2 (about 66 kPa), it can be seen clearly that the unit shaft resistance from lower cell to GL1 was significantly greater. The significant difference is due to the presence of the toe resistance for the pile segment below the lower cell, which resulted in a significant increase of strains recorded at GL1 and the influences of the first loading stage. It is noted that the unit shaft resistance of 66 kPa measured in the loading stage 2 is determined from the measurements of O-cell loads, not strain gages, with absence of the pile toe resistance (the lower cells were drained as shown on Figure 3). Therefore, only the unit shaft resistances determined from GL2 (about 300 and 150 kPa for pile stiffnesses of 73 and 186 GN, respectively) are reasonable to consider the pile shaft resistance in the sand layer. For two above shaft resistances, the unit shaft resistance of 150 kPa is close to the unit shaft resistances estimated from the standard penetration test results and thus it is reasonable to be representative of the pile shaft resistance in the sand layer.

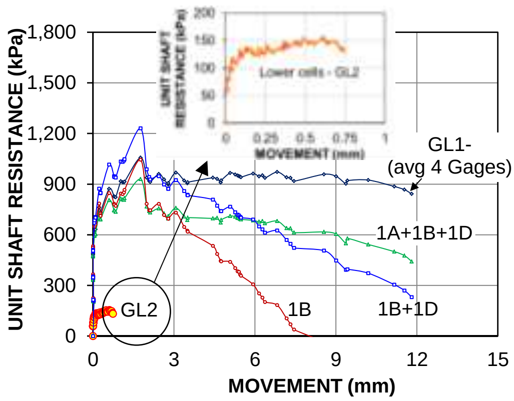

Figure 11. Unit shaft resistances versus movements computed basing on the pile stiffness of 186 GN.

6 CONCLUSIONS

The measurements of two-level bidirectional load test on the large-diameter long bored pile at the Cao Lanh cable-stayed bridge in Mekong delta of Vietnam have been presented. The analysis shows that the shaft resistances evaluated basing on the measurements of strain gage level GL1 is not reliable due to presence of the toe resistance. The lower cell level should be placed close to the pile toe to measure only toe resistance, instead of including the shaft resistance with one installed strain gage. Moreover, the shaft resistance between two cell levels evaluated from the loading of the upper cell level is not also reliable due to being significantly influenced by the loading of the lower cell level. The unit shaft resistance of the pile in sand layer determined is about 150 kPa.

7 REFERENCES

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