Finite Element Modelling of a Bidirectional Pile Test in Vietnam

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ABSTRACT: Static loading test on single piles for verification is commonly required, yet very expensive and difficult to perform, especially for the large-diameter bored piles. The bidirectional test, also-called Osterberg cell test, is nowadays very common in Vietnam. The Finite Element Method (FEM), which is a reliable tool for simulating loading tests, can also be used to model a bi-directional pile test. In this paper, FEM is used for modelling a bidirectional test on a 2.5m-diameter, 80m long bored pile at the Cao Lanh cable-stayed bridge in the Mekong Delta, Vietnam. The FEM results are compared with the monitored data obtained from the bi-directional test. The comparison showed that FEM can be an effective and reliable tool in this case. The FEM is performed using PLAXIS 2D.

KEYWORDS: Numerical analysis, Bidirectional test, Axial bearing capacity, Single pile, FEM

1. INTRODUCTION

FEM has been used to simualte coventional pile loading tests and predict the axial bearing capacity of piles. Predicting the axial bearing capacity of piles is always a challenging task. Therefore, static loading tests on single piles must be done for verification in the detailed design phase. These tests are very expensive and difficult to be carried out, especially for the large-diameter bored piles. The bidirectional test, also called Osterberg cell, is nowadays common in Vietnam. The acceptance of numerical analyses in geotechnical problems is growing and the finite element method, FEM, is more and more commonly applied in foundation design. FEM cannot replace the loading tests, yet it is a reliable tool for simulating loading tests on a single pile. In this paper, FEM is used for back-calculating a multi-level bidirectional test of a 2.5m-diameter, 80m long, bored pile at the Cao Lanh cable-stayed bridge in the Mekong Delta, Vietnam. The results from FEM analysis using PLAXIS 2D is then compared with the monitored data.

2. TESING PILE

2.1 Project

The Cao Lanh cable-stayed bridge, a part of the Central Mekong Delta Connectivity project (CMDCP), was constructed over the Tien River, Figure 1. The bridge has a 350 m main span length and 150 m wide side spans. The maximum height above high water level is 35.7 m. The main bridge is supported by 2.5 m diameter bored piles with a length ranging between 85 and 115m.

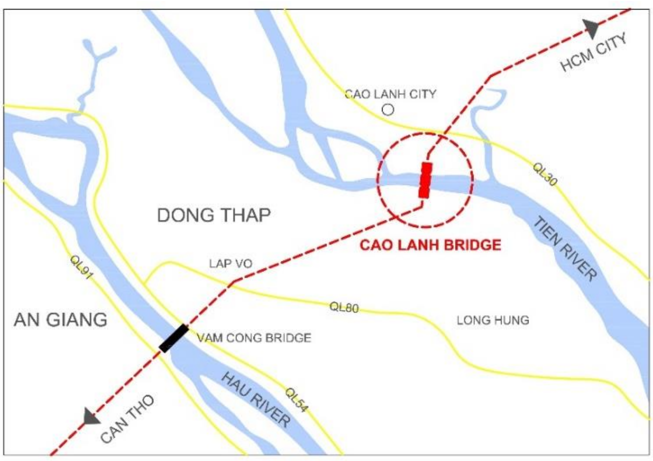


Figure 1 Project location

Four test piles, two at the pylons and two at the tie-down piers, were constructed prior to mass pilling work. In this paper, only Test Pile P20 is discussed. Due to a high working load and the complicated site conditions, performing a conventional static and dynamic loading test, which are commonly used in Vietnam, were impractical. The bi-directional test, was therefore performed.

2.2 Test pile P20 description

Test Pile P20 was completed on October 19, 2014. The pile has been excavated to 73.6 m depth (elevation -82.1 m), under bentonite slurry. A 2640 mm O.D. temporary steel casing (2560 mm I.D) was then installed to a depth of 31m depth below the mudline. A drilling bucket was used for drilling the pile, and de-sanding for cleaning. After the pile was approved for concrete placement, a reinforcing cage with two attached O-cell assemblies was inserted into the excavation and temporarily supported by the steel casing. The final depths were 60.1 and 72.1 m below the mudline; the lower O-cell assembly was placed at the pile toe. Concrete was then delivered into the pile through a 300 mm O.D. tremie pipe until the top of the concrete reached an elevation of +3.0 m, 5.5 m above the mudline. On November 9, 2014, the pile toe was grouted and 15 days later the pile was tested.

Figure 2 shows the soil profile and configuration of the instrumented pile at the site. The soil profile consists of an around 46 m thick deposit of alternating layers of loose silty sand, very soft sandy silt, medium stiff and very stiff clay, underlain by compact silty sand, becoming very dense at 60 m depth.

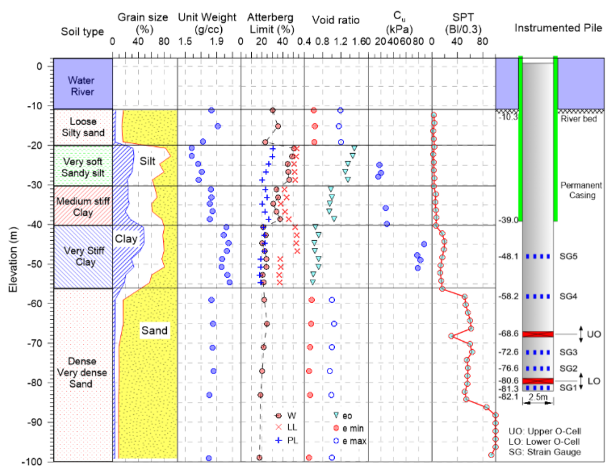


Figure 2 Geological profile and the instrumented pile

Pile Instrumentation: Each O-cell assembly consisted of two 670-mm-diameter hydraulic jacks, located at 1.50 m and 13.50 m above the pile base, respectively. A total of five strain gage levels, each with 2 pairs of gages (Geokon model 4150), diametrically opposed were installed.

2.3 Testing procedure

The test was carried out in the following stages:

- STAGE 1: In the first stage, the lower O-cell assembly was pressuriezed in 16 nominally equal increments, applied manually, to a maximum bi-directional load of 12.4 MN pushing against the combined shaft resistance and toe bearing of the pile length below the lower O-cell level, using the shaft resistance above as reaction. The testing was started by pressurizing the O-cell in order to break the tack welds that hold them closed (for handling and for placement in the pile) and to form the fracture plane in the surrounding the base of the lower O-cell. After maximum test load was achieved, unloading was performed in eight decrements till zero.
- STAGE 2a: The upper O-cell assembly was pressurized in 12 nominally equal increments, to a maximum bi-directional load of 6.2 MN. The shaft resistance at the upper part of the pile was used for reaction. The lower O-cell was left free to drain (no load transfer through the lower O-cell to toe resistance).
- STAGE 2b: After the maximum load test of 6.2 MN at the upper cell is reached in Stage 2a, the lower O-cell hydraulics were closed off and the loading at the upper O-cell is continued, to evaluate the shaft resistance characteristics of the pile section above the upper O-cell, using the shaft resistance bellow and the toe resistance as reaction. The upper pile part was loaded in 10 additional increments to a max bi-directional load of 14.5 MN. The loading was halted after the maximum test load has been achieved and the upper O-cell was then unloaded in six decrements.

The testing procedure for Test Pile P20 is shown in Figure 3. The load increments were applied using the quick load test method (ASTM D1143). Each successive load increment was held constant for fifteen minutes. All records of pressures and movements were electronically measured and recorded on a data collector.

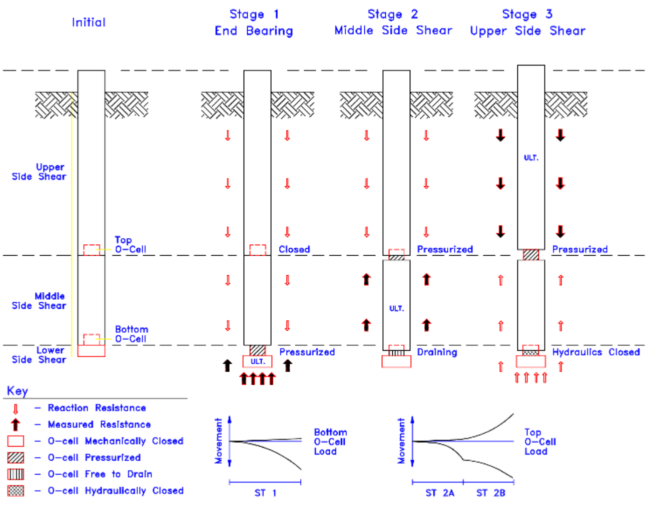


Figure 3 Loading test procedure

2.4 Test results

The load-displacement of test pile during testing procedure is presented in Figures 4 and 5. An Equivalent Head-down Load-movemenr curve is presented in Figure 6, as determined directly from the load-movements measured at the O-cell by combining the downward measurements of Stage 1 with the upward measurements of Stage 2.

For a 16.1 MN pile head load, the test data indicate that this pile would move approximately 9.5 mm, of which 8.4 mm is elastic compression additional to that measured in the test (caused by the compression in the piles for transferring load at the pile head to the O-cell level).

For a 24.2 MN pile head load, the test data indicate that this pile would move approximately 17.3 mm, of which 13.5 mm is additional elastic compression.

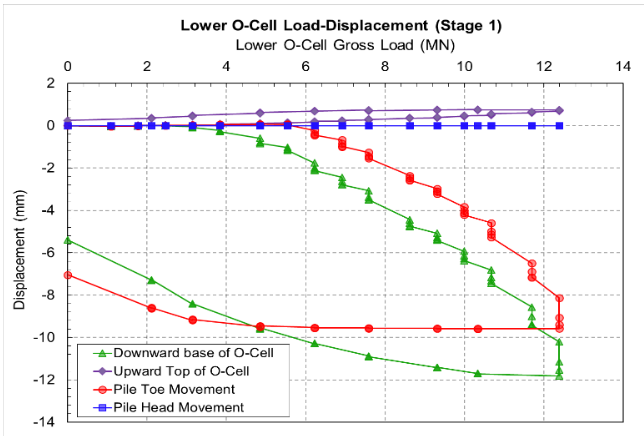


Figure 4 Lower O-cell load-displacement, Stage 1

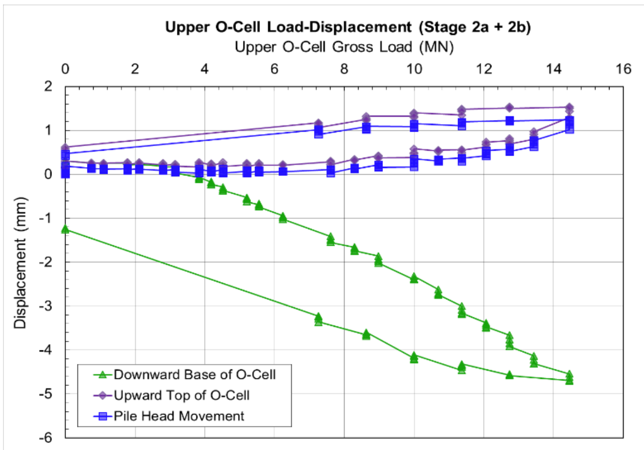


Figure 5 Upper O-cell load-displacement, Stage 2

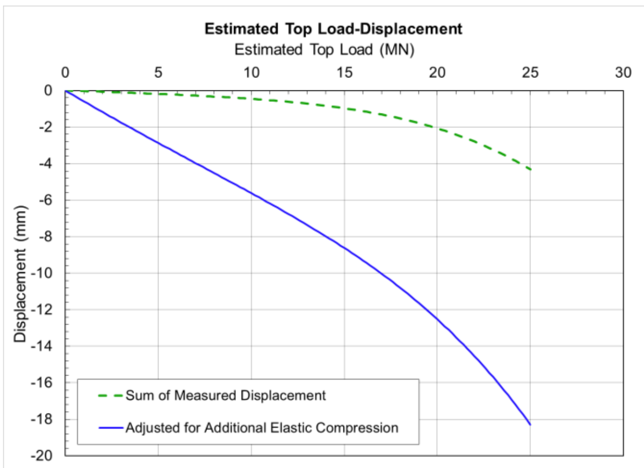


Figure 6 Equivalent Head-down Load-Movement

3. FEM ANALYSIS

Plaxis 2D is used for modeling the axi-symmetrical problem. In the analysis, the Hardening Soil (HS) was used to model the soil behavior. For boundary value problems that involve a mixture of loading and unloading stress paths, such a model is required because it captures all the general facets of soil behavior. Soil is essentially a non-linear material for almost all operative stress and strain levels encountered in pile testing. Ideally the HS-small models should be used but, in this work, we will begin with HS model.

The soil material parameters are summarized in Table 1. These parameters are related to the HS model. The single pile is model using the "Continuum-Solid" approach. This is important and necessary because the actual geometrical shape is required and essential. By doing so, the correct Toe Area and Perimeter Area of the pile is captured. A "Dummy Plate" is inserted along the length of the pile. This is to facilitate the extraction of output at the completion of the analysis. Displacements can be read directly. The development of Axial Forces due to the various sequence of loading in a bi-directional test can also be extracted but, has to be corrected due to the scaling down of mechanical parameters when using the "Dummy-Plate" approach.

The O-cells were modeled and the FEM numerical equivalence is the usage of uniformly distributed load boundaries applied in opposition at the cell-pile interfaces. When the O-cells are activated in the simulation, the loads are activated in the relevant direction. The volume representing the O-cell is replaced with a "Soft-Dummy" zone.

The material parameters are summarized in Table 1. The bored pile is modeled by volume elements with non-porous linear elastic material, with a modulus of 22.7E6 kN/m^2. The O-cell "Dummy material" has a modulus of 1000 kN/m^2. The element mesh is shown in Figure 7a.

The jacking cells are simulated by line loads placed at the cell levels, i.e. Elev. -68.6m for the upper cell, and Elev. -80.5m for the lower cell. Each cell is modeled by a pair of line loads, one upward and another downward, see Figure 7b.

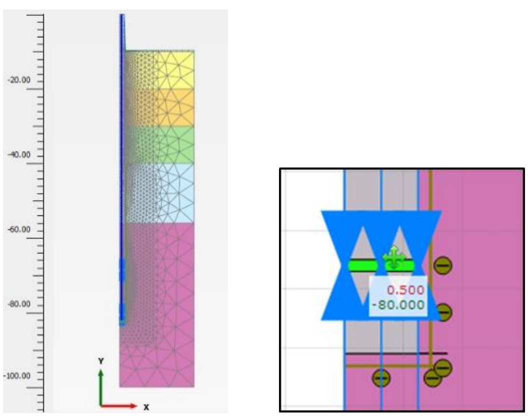
The staged construction analysis was performed with four calculation phases:

- STAGE 1a: Loading to 12.4 MN
- STAGE 1b: Unloading
- STAGE 2a: Loading to 6.2 MN
- STAGE 2b: Loading to 14.5 MN

In the first phase, Stage 1a, the lower cell was active with a load of 12.4MN. The load was released to 0 MN in Stage 1b, Figure 8. In the two following phases, Stages 2a and 2b, the upper cell was active, with a load of 6.2 MN and 14.5 MN respectively, Figure 9.

Table 1 Material parameters

Parameter		Loose silty sand	Sandy silt	Medium Stiff Clay	Very stiff clay	Very dense sand
Drainage type		UDR(A)	UDR(B)	UDR(B)	UDR(B)	UDR(A)
γ_{unsat}	[kN/m³]	17.0	17.0	18.5	20.0	17.0
γ_{sat}	[kN/m³]	20.0	17.0	18.5	20.0	20.0
$c'/s_{u,ref}$	[kPa]	1.0	20	30	80	0.0
φ'/φ_u	[°]	25	-	-	-	33
ψ	[°]	0	-	-	-	3
ν_{ur}/ν'_{ur}	[-]	0.2	0.2	0.2	0.2	0.2
E_{50}^{ref}	[MPa]	15	16	24	64	200
E_{oed}^{ref}	[MPa]	15	16	24	64	200
E_{ur}^{ref}	[MPa]	45	48	72	192	600
m	[-]	0.5	0.5	0.5	1.0	0.5
p_{ref}	[kPa]	100	100	100	100	100
K_0^{nc}	[-]	Auto	Auto	Auto	Auto	Auto
R_f	[-]	0.9	0.9	0.9	0.9	0.9
$\sigma_{Tens.}$	[kPa]	0.0	0.0	0.0	0.0	0.0



(a) (b)

Figure 7 (a) Element mesh, (b) Cell model

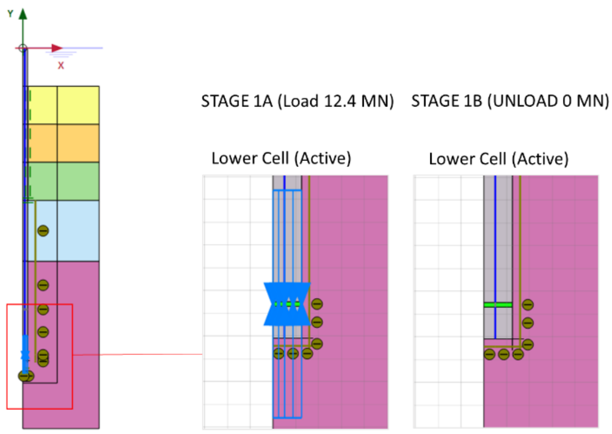


Figure 8 FEM model, Stage 1a and 1b

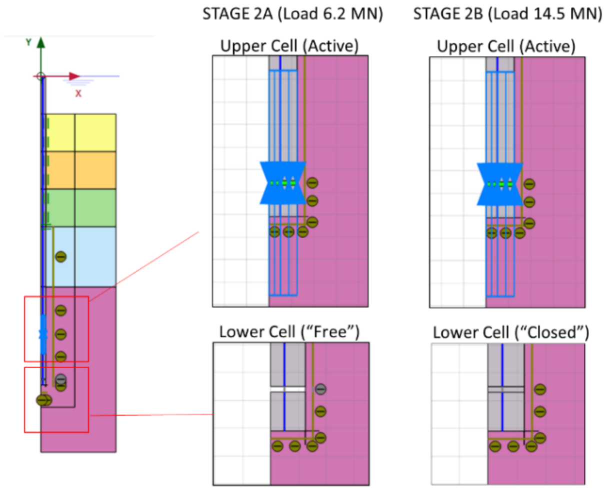


Figure 9 FEM model, Stages 2a and 2b

4. COMPARISON OF MONITORED AND FEM RESULTS

The results obtained from the FEM analysis are compared with the loading test data only for Stage 1, i.e loading the lower cell to 12.4 MN and unloading to 0.0 MN. The comparison is shown in Figure 10, and indicates generally a good agreenent beteen the monitored and FEM results.

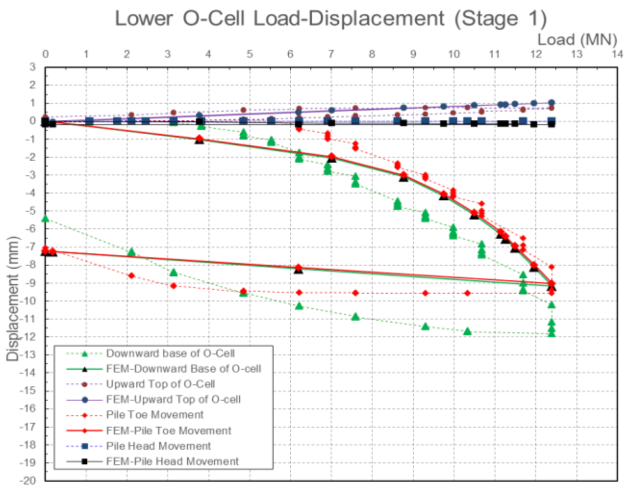


Figure 10 Comparison of monitoring and FEM

The FEM downward movement at pile toe, and that at the lower cell base are close because these two points are only 1.5m away from each other. The difference is only the elastic compression of this 1.5 m pile length, which is very small. These two lines are close to the monitored downward movement value at the pile toe.

The monitored downward movement at the lower cell base is however about 2.5 mm bigger than that at the pile toe. This can be explained by the gaps formed between the cell plate and concrete during the construction process. Under the loading, the gaps are first narrowed. This made downward movement at the lower cell base much bigger than that at the pile base.

For the upward movement, a good agreement can be observed between the monitored and the FEM values, see Figure 10.

5. FEM ANALYSIS TO FAILURE

In Test Pile P20, the maximum test load is not big enough, and the pile movements are therefore too small. The shaft resistance is far from being fully mobilised and the toe resistance is hardly engaged. Since PLAXIS can capture the realistic behaviour of the pile as shown in Figure 10, it can be used to predict the ultimate pile load that is not achieved in the load test. In order to obtain the ultimate load, a load simulation is applied on the pile top.

The load-displacement curve received from the simulation is shown in Figure 11. It can be seen that the ultimate load is approximately 49.0 MN, which is determined by the double-tangent method. This ultimate load is three times larger than the maximum load performed in the test, i.e. 14.5 MN.

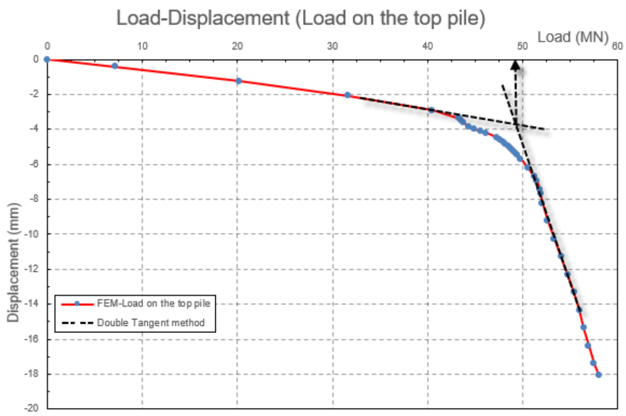


Figure 11 FEM analysis to failure

6. CONCLUSIONS AND SUGGESTIONS

The Finite Element Method (FEM), although it cannot replace the loading tests, it is useful for back-calculating the results of a bidirectional test on single piles, or for simulation of slightly different piles at a site, e.g., slender or wider, longer or shorter. The paper shows an example of back-calculation of a test for such use

FEM has been used to simualte coventional pile loading tests. With the FE model given in the paper, a bi-directional test can also be simulated very well. For the tests in which the maximum test load is not big enough, FEM analysis allows simulation of a response beyond that observed.

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