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50 MN Statnamic test on 2.0 m diameter instrumented bored pile

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ABSTRACT

A case history of two preliminary instrumented 2.0 m diameter, 63 meter long test piles at two pylon positions of the 500 meter main span single cable plain type cable stayed bridge.

The pile was constructed without rock socket with a working load of 25,000 kN. The pile is instrumented with resistance wire strain gauges at 9 levels with three gauges mounted per level and piezoelectronic accelerometers were installed at two levels. Statnamic tests were performed at twice working load, 50,000 kN in order to evaluate the design parameters in relation to the ultimate shaft resistance and end bearing.

The behaviour of pile settlement and elastic shortening of pile under applied load shall be discussed. The load-movement relation, distribution of axial load along the pile and the load transfer characteristics of the pile will be presented and discussed. The pile performance from the load test is compared with the design parameters used. Statnamic test provides reasonably good data for the evaluation of the pile performance for large loads and in the marine environment.

1 INTRODUCTION

The instrumented test pile discussed was carried at the eastern pylon pier location of the 500 m centre span Cable Stayed Bridge. The foundation of the pylon pier comprises 29 number of 2.0 m diameter bored piles.

2 PROJECT

The Senai-Pasir Gudang-Desaru Highway project undertaken by the government of Malaysia connects Senai, Pasir Gudang and Desaru and traverses through varied terrain. The proposed 1.708 km bridge forms part of highway connecting Pulau Juling to Tanjung Penyambung over Sg. Johor.

3 DESIGN PHILOSOPHY

The geotechnical capacity of the bored piles was estimated using a factor of safety of 2 for frictional resistance and 3 for end bearing. The static ultimate geotechnical compression capacity of bored pile has been estimated for granular soils using the following relationship:

$$Q_{ult} = a N_s A_s + b N_b A_b \quad \text{Eq. (1)}$$

(a =1 for less than 22.5m and 2 for depths greater than 22.5m; b ≤133 for gravel & coarse sand and ≤100 for fine sand and non-plastic silt)

$$Q_{allowable} = (a N_s A_s)/2 + (b N_b A_b)/3 \quad \text{Eq. (2)}$$

Where:-

Q_{ult} is the ultimate pile capacity;

$Q_{allowable}$ is the allowable pile capacity;

A_s is the surface area of pile shaft;

N_s is the average SPT 'N' value along pile shaft;

N_b is the average SPT 'N' value below the pile toe;

The limiting value for unit skin resistance, f_{su} , is 150 kN/m² for SPT 'N' average value of 75

The limiting value for unit base resistance, f_{bu} , is 10,700 kN/m².

The soil profile within the site indicates a soft clay for depth up to 22.5m while beyond this depth the soil is medium dense sand and very stiff silt. The relationship for unit skin friction, $f_s = 2N$ for residual soils in Malaysia and Singapore has been put forward by researchers based on instrumented pile tests carried out in this region for bored piles of different diameters and lengths. The value of 'a' in equation 1 for depth up to 22.5 m ($f_s=1N$) is used.

The Standard Penetration Test blow count of SPT 'N' of 75 has been referenced in order to limit the unit skin friction to 150kN/m^2 ($f_{su} = 2N$).

4 INSTRUMENTATION

The instrumented pile test programme was designed to assess the contribution of the skin frictional resistance and the end bearing resistance to the load carrying capacity of the bored cast-in-situ test pile. The instrumentation comprised nine (9) levels of resistance wire strain gauges with three(3) numbers at each level. In addition two (2) piezoelectronic accelerometers were installed at two(2) levels as shown in Figure 1. A dummy strain gauge was also provided outside of the pile. In the test pile, each strain gauge was mounted to the main reinforcement bar at the designated level by welding. The signal cables from the strain gauges were led along the reinforcement bars with cable ties to the top of the pile where readings are obtained from a data logger. The monitoring of strain gauges was carried out using an independent data logger which automatically measured and recorded the required data during load testing of the test pile. A portable computer was connected to the data logger to enable display of readings, data retrieval and re-programming.

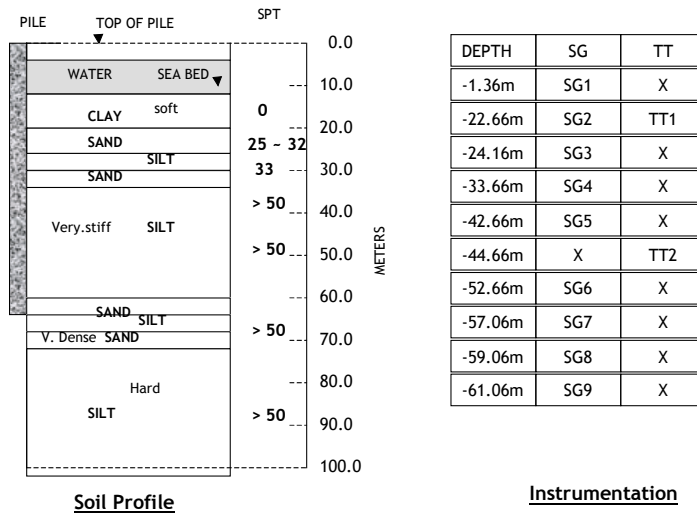


Figure 1 : Soil Profile and Instrumentation Levels for Pile PIE

5 STATNAMIC TEST

The Statnamic load testing method was developed in 1988 and is now widely used in various infrastructure projects. The load testing device is similar in concept to a compressed air-jack. High pressure gas forces separate the Statnamic piston and cylinder with the piston acting downward on the foundation element (test pile) and the cylinder acting upward against a reaction mass. The gas that powers the load test is produced by the burning of a solid fuel propellant. The load duration of a Statnamic test is usually about 120 milliseconds. The instrumented test pile P1E was tested by using the Statnamic load test

method using set-up shown in Figure 2. In the set-up used, the test load was applied using compressed high pressured air jack (600 tons capacity). The applied load was measured by a calibrated load cell mounted at the pile top. The loading from a Statnamic load test is of a much longer duration than that from a pile hammer or drop hammer, so the foundation moves downward almost as a single unit and may be analyzed as if it were a rigid body. Therefore the Statnamic load test does not rely on a numerical model in the response of the foundation. Instead, it directly develops a load-displacement curve, and thus is conceptually similar to a static load test, except that it is performed rather quickly.

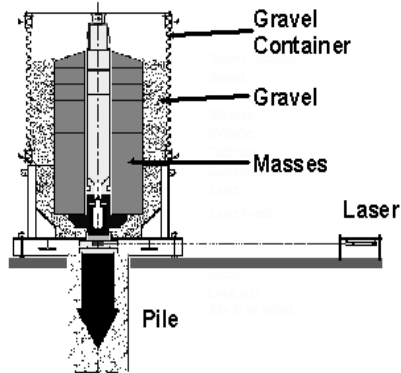


Figure 2: Schematic of Statnamic set-up

Data reduction methods attempt to account for inertial and damping forces. Since the static response of the soil differs from the dynamic response, so the load settlement curve obtained from Statnamic load test can differ from that obtained from a static load test. The data reduction attempts to account for this difference, along with the inertial effects in the foundation. In a purely homogenous fine grained sub-soil strata where pore pressure can influence the Statnamic Test, the pile carrying capacity is not directly affected but the settlement predicted would be underestimated compared to a Slow Maintained Load Test.

6 TEST RESULTS AND INTERPRETATION

6.1 Pile Settlement

A maximum Statnamic load of 52.03 MN was applied during the load test. The maximum static resistance was 51.56 MN. The gross displacement of the pile head during the test was 16.50 mm. The recovery from this displacement was 15.0 mm leaving behind 1.4 mm of permanent settlement. The test pile moved with a maximum velocity of 0.6m/s.

6.2 Load Distribution and Transfer Characteristics

The force from the explosion generated a downward movement in the foundation, which was measured using the strain gauges. The data obtained from these instruments was plotted directly as load distribution curves similar to those obtained from a static load test. The load distribution curves for the test cycles are based on strain-load computations based on the measured changes in strain gauge reading and estimated pile properties (steel content, cross-section areas and Modulus of Elasticity).

Load transferred (P) at each level is calculated as follows:-

$$P = e (E_c A_c + E_s A_s) \quad \text{Eq. (3)}$$

Where,

e is the average change in strain gauge readings;

A_c is the cross-sectional area of cement grout;

E_c is the Young's Modulus of Elasticity grout;

A_s is the cross-sectional area of steel reinforcement;

E_s is the Young's Modulus of Elasticity in steel = 200kN/mm².

The Young's Modulus of Elasticity in concrete, E_c , was estimated with the aid of the strain gauge results at level 1 and the pile top loads. For each stage of loading, E_c is back-calculated by assuming that the loads at the strain gauge level 1 were equal to the applied load at the pile top. However back calculated E_c may vary with the pile depth and the time lapse in the pile, the relationship of which is not covered in this load test programme. The complication of measuring compression strain in composite reinforced member is worth to be noted as well. Hence the computed results are true only within the above limitations. Figure 3 presents the strain reading at all levels with time.

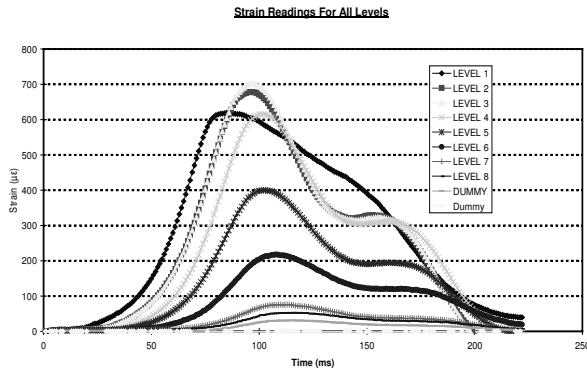


Figure 3 : Strain Readings for All Levels

The difference between the loads at any two levels represents the skin frictional load carried by the portion of pile between the two levels. The distribution curves obtained depict the normal trend of decreasing axial loads with depth. It is clearly seen in Figure 3 that the load transfer distribution at the nine levels indicates that the upper portion of the pile develops the highest load transfer and the bottom-most portion of the pile develops the least load transfer. Figure 4 shows the depth of gauges versus the load transfer and distribution. It is clearly evident that the maximum load transferred occurs in the shaft for all percentages of loads up to 2.0 times the working load.

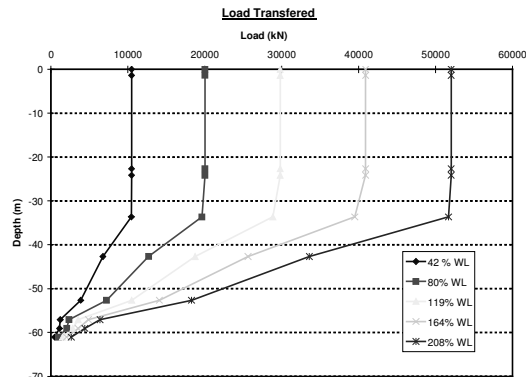


Figure 4 : Plot of Load Transferred

Figures 5 and 6 indicates the load distribution versus pile top displacement. The displacement characteristics shows equal displacement at all levels of instrumentation.

Similarly, the characteristics indicate that the shaft friction is mobilised between level 4 to level 8 at twice working load has exceeded limiting designed value of 150 kN/m².

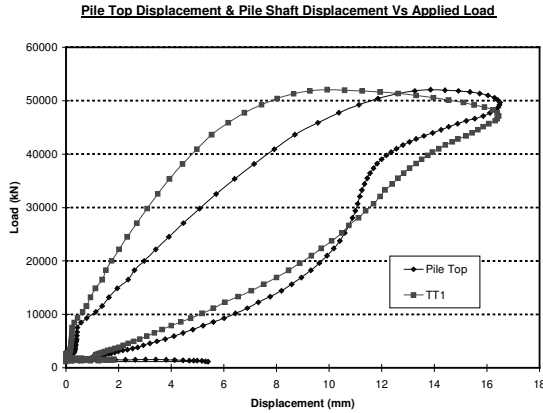


Figure 5 : Pile Top Displacement & Shaft Displacement versus Applied Load

Figure 6 indicate the mobilised unit skin friction versus applied load. The figure demonstrates that the upper portion of the skin mobilises almost twice the unit skin friction than that of the lower portion. Figure 7 shows that the displacement is equal at all levels of the instrumentation, indicating that skin friction has been mobilised with very little base resistance.

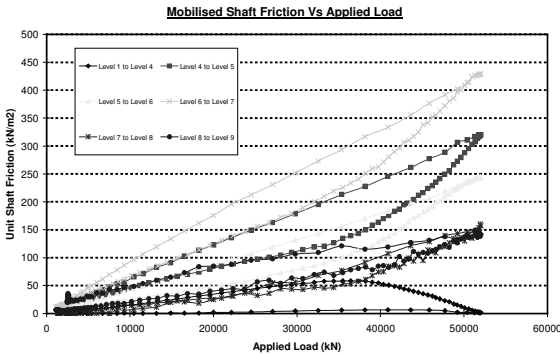


Figure 6 : Mobilised Skin Friction versus Applied load

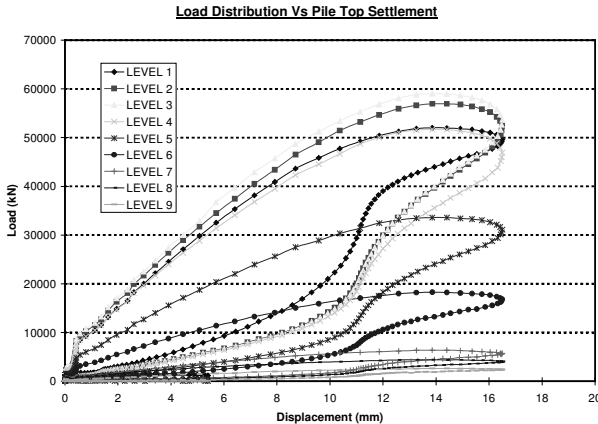


Figure 7 : Load Distribution versus Pile Top Settlement

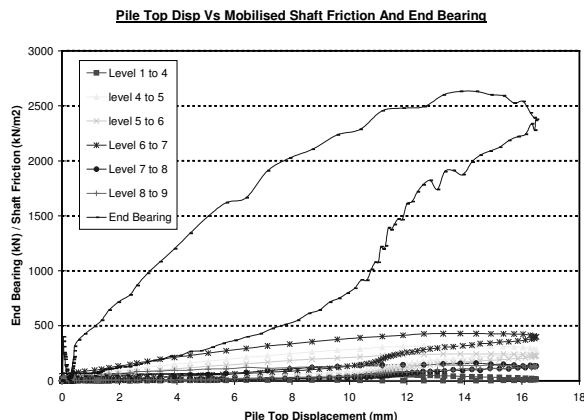


Figure 8 : Pile Top Displacement versus Mobilised Skin Friction and End Bearing

From Figure 8, it can be seen that the mobilised unit shaft friction at all levels except between level 6 and 7 are below the limiting value of 150kN/m^2 at working load of 20000kN . The mobilised unit skin friction at twice working load exceeds the limiting value without compromising on the settlement criteria which is not greater than 10mm at WL.

7 CONCLUSIONS

The Statnamic load test on the instrumented bored pile clearly indicates that the load transfer is mainly on the pile shaft. The instrumentation results indicates good agreement with the initial design assumption which was based on a major friction pile mobilisation. The instrumented pile indicates that the Statnamic load test provides a representative behaviour of the pile tested. This study indicates that Statnamic load testing is a viable method for pile load testing. Furthermore in marine conditions it is would have been a mammoth task to carry out maintained load test.

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