Dynamic Pile Testing of Three Drilled Shafts on the Bridge over the Changuinola River in Panama.

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ABSTRACT

The Changinola River project contemplates the construction of a bridge which will replace the existing one, which had been used for over one hundred years exclusively for railroad traffic. The new bridge is a double span concrete structure and will have two lanes for cars and a pedestrian pass. It will be supported by reinforced concrete piers, with a diameter of 1.3 m (51 inches) and depths varying from 18 m to 20 m (50 feet to 66 feet), including a 3.5 m (11.5 feet) socket perforated into sound rock, underlying predominantly clean gravel soil, characteristic of those tributaries. Three of those piers were tested, on three different abutments. One of the piles had previously been submitted to an Osterberg Cell load test, so the presence of the cell had to be taken into consideration when analyzing the dynamic test data. A comparison of the results of the Osterberg Cell and of the Dynamic Load test could also be made. The impact system used for the tests was carefully designed using wave equation analyses, and consisted of a 265 kN (59.5 kips) steel ram falling from a maximum height of 2.5 m (8.2 feet). A 250 mm (10 inches) thick plywood cushion was used between the ram and the top of the pile. Details of the wave equation analyses are also shown.

INTRODUCTION

The Changinola River project in the *Bocas del Toro* province in Panama contemplates the construction of a bridge which will replace the existing one, which had been used for more than one hundred years. The new bridge is a double span concrete structure and will have two lanes for cars and a pedestrian pass. The foundation design calls for piers made of reinforced concrete with a nominal strength of at least 30 MPa (4.3 ksi), with a diameter of 1.3 m (51 inches) and depths varying from 18 m to 20 m (50 feet to 66 feet), including a 3.5 m (11.5 feet) socket drilled into sound rock, underlying predominantly clean gravel soil.

Three piles were selected for dynamic testing. Two of them were production piles which were later used in the structure (E-4-5 and E-3-14), and one was a test pile which had previously been subjected to an Osterberg Cell test (E-2-P).

The testing process consisted of four main phases:

1. Wave equation analyses were made using predicted and recommended soil parameters based on the available soil information. These analyses were made for determining the ram mass and drop height, so that a set of at least 2.5 mm (0.1 inch) was reached to insure full soil resistance mobilization, while making sure that the stresses along the pile were kept at a safe level. Since pile E-2-P had the Osterberg Cell installed approximately 3.5 m (11.5 feet) from the pile toe, a special wave equation analysis had to be made in this case, in order to take into account both the non-uniform pile characteristics and the reduced skin friction in the region of the cell.
2. A hammer system was specially constructed for the test, based on the results of the wave equation analyses.
3. The piles were cast with a specially reinforced extension to allow the installation of the gages. After the sensors were installed, a test sequence consisting of blows with increasing drop heights was applied.
4. The data from the blow corresponding to the maximum mobilized capacity was analyzed with the CAPWAP® program to confirm the results obtained in the field with the CASE method, and also to obtain the resistance distribution and other soil parameters. A non-uniform model was used for pile E-2-P, to take into account the presence of the Osterberg cell. The dynamic test results for this pile were compared with the results from the Osterberg cell.

A detailed explanation of each phase follows.

WAVE EQUATION ANALYSIS

Using wave equation analysis (Rausche et.al., 2004) it was possible to predict the pile set and stresses for blows of different ram weights using different drop heights. A simple drop hammer was simulated, with a ratio of kinetic to potential energy of 0.8. The driving system consisted only of a plywood cushion with the same cross-section area as the pile, placed between the ram and the top of the pile. The effect of changing the thickness of the plywood cushion was also investigated.

For piles E-4-5 and E-3-14, a simplified soil model was deemed sufficient. This model represented the soil as having two layers, the upper one with a triangular distribution, and the lower layer with constant resistance with a value 76% higher than the maximum resistance of the overlaying layer. Figure 1 below shows a diagram of the pile, hammer and soil model used.

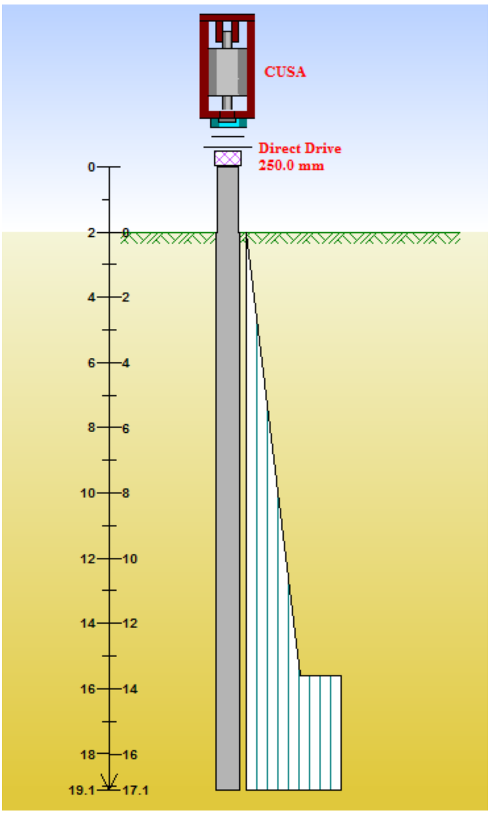

Figure 1: Pile, hammer and soil model for the uniform piles

An ultimate soil capacity of 14.7 MN (3300 kips) was used for the wave equation analyses. This is the specified capacity for pile E-3-14. The actual ultimate capacity for piles E-4-5 and E-2-P is 11.2 MN (2500 kips).

A non-uniform model was used for pile E-2-P, to take into account the presence of the 0.3 m (12 inches) thick, 0.4 m (16 inches) diameter Osterberg cell, with a top and bottom steel plate of 0.96 m (37.8 inches) diameter and 0.05 m (2 inches) thickness. A more elaborate soil model was used in this case, consisting of an upper poorly graded gravel layer, 12.5 m (41 feet) thick, with an average SPT N Value of 24, overlying a 3.5 m (11.5 feet) thick rock layer with a unit skin friction of 294 kPa (6.1 ksf) and end bearing of 3285 kPa (68.6 ksf). Figure 2 below shows a diagram of the pile, hammer and soil model used.

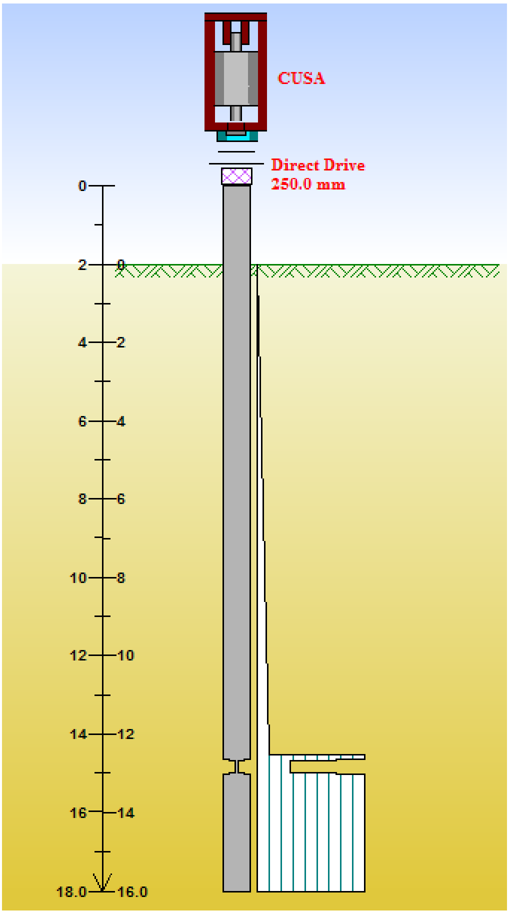

Figure 2: Pile, hammer and soil model for the non-uniform pile

Using the models described above it was possible to simulate several ram weight/hammer cushion thickness combinations, until a configuration was found that would work best in all cases. This consisted of a 265 kN (59.5 kips) ram with a wooden cushion 250 mm (10 inches) thick. The simulations indicated that a drop height of up to 2.5 m (8.2 feet) could be used without risking damaging the piles. For an ultimate load of 14.7 MN (3300 kips) the predicted compressive stresses were always below 19.4 MPa (2.8 ksi), and the tension stresses were below 4.6 MPa (0.7 ksi). For that load and drop height, the predicted set for the uniform piles was 10.7 mm (0.4 inches) and for the pile with the Osterberg cell the analysis predicted a set of 7.5 mm (0.3 inches). Those sets were above the minimum necessary to guarantee full mobilization of the soil resistance.

TEST PROCEDURE

A driving system was constructed, with a ram consisting of A-36 steel plates held together by #11 grade 60 steel bars, which totaled the necessary weight. A 60 tonne (590 kN – 132 kips) crane was used to lift the ram, which was guided by a steel frame surrounding the pile. Figures 3a and 3b below show the driving system used.

Figure 3a and 3b: Driving systems used in the tests.

The piles were constructed by excavating using a casing oscillator and a mechanical grab. Concrete was then poured after insertion of the reinforcing cage, while the casing was extracted using the oscillator as the concreting process progressed. The test piles had an extension sticking out of the ground, with special reinforcement in order to withstand the hammer blows. The sensors were installed on the smooth surface resulting from the extraction of the casing.

DYNAMIC TEST RESULTS

The tests consisted of blows with increasing drop heights. The data from the sensors were sent to a Pile Driving Analyzer® (PDA), which processed and recorded the signals after each blow. The sets for each blow were measured by a laser beam placed a sufficient distance from the pile to reduce disturbance by soil vibrations. The signals from the blows corresponding to the maximum mobilized resistance (the last blows applied, in all three cases) were analyzed using the CAPWAP program (Likins et.al., 2000). This is a signal matching program (system identification analysis or reverse analysis), which allows the determination of the soil parameters from the inputs of the measured force and velocity signals and the geometry and stiffness of the pile.

Table 1 below summarizes the most important results.

Table 1: Dynamic test results

Pile	Length (m)	Blow nr	Drop height	set/blow (mm)	Max Transf'd Energy (kJ)	CAPWAP results (MN)		
						Skin	Toe	Total
E-4-5	16.9	4	1.8	0.5	64.1	11.4	8.5	19.9
E-3-14	21.4	3	1.9	0.1	54.1	8.8	7.0	15.8
E-2-P	16.5	3	2.5	1.0	41.9	7.8	4.6	12.4

COMPARISON WITH OSTERBERG CELL RESULTS

Pile E-2-P had been previously tested using the Osterberg method. A detailed description of the Osterberg cell (O-cell) method (FHWA, 2006) is beyond the scope of this paper. In summary, the two O-cells were pressurized in 21 loading increments, holding each load increment constant for eight minutes, to 72.39 MPa (10,500 psi). This resulted in a bi-directional gross O-cell load of 13.10 MN (3000 kips). The loading was stopped after the 21st increment because the anticipated ultimate loads had already been exceeded and the nominal O-cell capacity had been reached. The maximum upward applied net load (gross O-cell load minus the buoyant weight of the pile above the cell) to the upper side shear was 12.71 MN (2800 kips). At this loading the upward movement of the O-cell assembly was 5.36 mm (0.2 inches). The maximum O-cell load applied to the combined end bearing and lower side shear was 13.10 MN (3000 kips). At this loading the average downward movement of the O-cell assembly was 11.71 mm (0.46 inch).

The results of an O-cell test can be used to estimate the top-loaded load-settlement curve. Based on this curve it was possible to estimate a settlement of 2.8 mm (0.11 inches) for a top loading of 7.4 MN (1660 kips - roughly equivalent to the maximum service load of the piles). CAPWAP estimated about 3 mm (0.12 inches) for the same load (a difference of less than 7%).

For a top loading of 14.7 MN (3300 kips) the adjusted O-cell test data indicates a settlement of 6.4 mm (0.25 inches). The dynamic test of pile E-2-P did not reach this mobilized load. For the maximum mobilized capacity of about 12.4 MN (2800 kips) CAPWAP is indicating a settlement of 7.3 mm (0.29 inches), whereas the O-cell estimate is slightly more than 5 mm (0.2 inches) for this load. However, for pile E-3-14 the CAPWAP predicted settlements for 12.4 MN (2800 kips) and 14.7 MN (3300 kips) loads were respectively 4.7 mm (0.18 inches) and 6.0 mm (0.24 inches), therefore less than the O-cell predictions. The CAPWAP predicted settlements for pile E-4-5 were even smaller.

CONCLUSIONS

Three drilled shafts, 1.3 m (51 inches) in diameter and with lengths ranging from 16.5 m to 21.4 m (54.1 to 70.2 feet), including a 3.5 m (11.5 feet) rock socket, were dynamically tested. The driving system consisted of a 265 kN (59.5 kips) ram free falling from heights varying from 0.45 m (1.5 feet) to 2.5 m (8.2 feet). The driving system was designed using wave equation analyses so as to guarantee full soil resistance mobilization with safe stresses.

Total capacities between 12.4 MN (2800 kips) and 19.9 MN (4500 kips) were mobilized, with small permanent sets (0.1 mm to 1 mm – 0.004 inches to 0.04 inches), indicating that the actual ultimate resistances are still higher. The maximum mobilized capacities were satisfactory when compared to the required ultimate capacities.

One of the piles had previously been subjected to an Osterberg cell load test. The top-loaded load-settlement curve obtained from the Osterberg cell test data indicated a higher maximum mobilized load, and a stiffer pile-soil behavior at higher loads, when compared to the CAPWAP simulated load-settlement curve for this pile. However, for the two piles not tested with the O-cell the results of the top-loaded load-settlement curve derived from the Osterberg cell test data are showing a less stiff pile-soil behavior than the corresponding CAPWAP simulated load-settlement curves. The discrepancy in the results can therefore be attributed in part to the fact that the construction of the load-settlement curve from the Osterberg cell data does not take into consideration the reduction in pile stiffness caused by the cell itself, but instead assumes a uniform pile. It should be noted that the cell had not been grouted for the dynamic load test.

REFERENCES

Federal Highway Administration (FHWA) (April 2006). "Design and Construction of Driven Pile Foundations." *Publication No. FHWA NHI-05-043, Reference Manual* (Volume II): Chapter 19

Likins, G.E., Rausche, F. and Goble, G.G. (2000). "High strain dynamic pile testing, equipment and practice." *Proceedings, VI Conference on the Application of Stress Theory to Piles*, Niyama & Beim (eds), Balkema, Rotterdam: 327-333.

Rausche, F., Liang, L., Allin, R.C. and Rancman, D. (2004). "Applications and correlations of the wave equation analysis program GRLWEAP." *Proceedings, VII Conference on the Application of Stress Wave Theory to Piles*, The Institution of Engineers Malaysia, Petaling Jaya, Selangor, Malaysia: 107-123