CASE STUDY OF THREE BIDIRECTIONAL STATIC LOAD TESTS FOR BORED PILES AT SABANA COSTA RICA

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ABSTRACT

Three bidirectional static load tests (BDSLT) were executed on bored concrete piles, as part of the permanent foundations for two buildings sites located at the Sabana, San Jose, Costa Rica, separated at a relatively short distance of about 1 km. Unit skin friction values obtained for both sites, principally for the "Lahar" layer show that in general, friction pile capacity from the tests is higher than anticipated per static calculations from soil borings geotechnical parameters correlations. Results from the tests were also compared with dynamic load tests (DLT) executed at other two sites in the surrounding area, and with a Finite Element (FE) Analysis that model soil-pile interaction for one of the projects. On the other side, some construction issues were detected on one of the sites, which negatively affected the end bearing capacity, but being that pile friction was higher than anticipated, the required overall pile capacity was accomplished. The BDSLTs results of these tests help to better understand the geotechnical behavior of the soil layers in the Sabana. Moreover, they also show that is possible to optimize the design and construction of other bored pile supported buildings and structures on the zone.

Keywords: Bored pile, Bidirectional Static Load Test, Dynamic Load Test, Finite Element Analysis, single pile analysis

1 PROJECTS DESCRIPCTION

La Sabana is located East of San Jose city and is currently the epicenter of several vertical developments in Costa Rica. The authors have been involved in several projects in the area, including the execution of Bidirectional Static Load Tests and Dynamic Tests on piles to estimate and verify the expected pile capacity on each project. Buildings that are analysed in this paper are relatively near each other, within a radius smaller than 1 km, and are still at different stages of construction.

Location of each project could be seen on Fig. 1, as follows:

1. Universal Tower: 21 stories building, with columns over a conventional pile foundation. Wide separation between columns yield high forces on piles of about 22 MN.
2. Double Tree: 38 stories building with a pile raft foundation. Piles are 1.0 m diameter, depths of 15 and 20 m with required capacities of 13 MN and 16 MN.
3. Cosmopolitan Tower: 20 stories building with a pile raft foundation. Piles are 30 of 0.8 m diameter and 15 m depth, and 38 piles of 1.0 m and 20 m depth. Required capacity for the 0.8 m pile, which was tested, is 8 MN.
4. Secrt Tower: 37 stories building with a pile raft foundation. Piles are 138 of 1.0 m diameter and 20 m depth, with a required capacity of 8 MN.

2 GEOTECHNICAL PROFILE

For the case of the project Double Tree, and in order to characterize the typical subsurface conditions around the area of interest, three (3) different type of tests were conducted on the site of the proposed project area, the selected tests were: four (4) DMT (Marchetti Dilatometer Testing, ASTM D6635-01) from 17.6 to 19,6 ,6 m depth, eleven (11) PMTs (TEXAM Pressumeter, ASTM-D4719-07) executed at 18.0 to 39.2 from top of sounding and two (2) MASW (Multichannel Analysis of Surface Waves).

Fig. 1. Location of the projects referenced on this article: 1. Universal Tower, 2. Double Tree, 3. Cosmopolitan Tower, 4. Secrt Tower Ref. Google Map and self-elaboration.

The PMT campaign was programmed for the determination of the bearing capacity and deformability of the existing materials below the maximum explored depth of the DMT soundings. Each MASW is the result of the seismic implantation of 12 vertical geophones of low frequency (4.5 Hz) distributed along 22 meters with intervals of 2.0 meters each. Source-geophone distances of 4 and 26 meters were considered. As a wave source, a 4.5 kilograms hammer and a striking plate were used.

Also, six (6) core drilled borings from 25 to 45 m depth were carried out with a double purpose: a) To physically describe the different layers found and, b) To have access for the PMT probe in order to test at different elevations. No SPT sampling and i.e. direct recovering of soil samples was executed since DMT soundings was the preferred procedure for determining resistance parameters and solid description in the softer layers. DMT uses a blade with a membrane, introduced without soil removal.

The soil profile determined by the DMT soundings was defined by its geo-mechanical behavior (ID), and its considered homogenous since the borings show the same tendency with materials of sandy silt textures up to a depth of 13.0-16.0 m, and a subjacent material detected up to a depth of 17.6-19.6 m characterized by silts and clayey silts.

Table 1. Soil profile for Double Tree and for the referenced sites. Ref. Geotechnical studies for the sites.

Depth (m)	Layer ID	E (kPa)	c' (kPa)	φ'	ψ	K_0	γ (kN/m^3)	K_{0NC}	ν
0-2,2	Layer I	14440	0	39	9	0.78	18.4	0.36	0.26
2,2-4,0	Layer II	75630	10	41	11	0.93	21.3	0.36	0.26
4,0-9,0	Layer III	32210	0	33	3	0.59	20.6	0.47	0.34
9,0-12,6	Layer IV	58610	30	33	3	0.70	22.0	0.45	0.31
12,6-20,0	Layer V	89810	50	35	5	0.79	22.5	0.44	0.31
20,0–27,0	Layer VI	138053	50	38	8	1.15	20.0	0.38	0.20
27,0-34,0	Layer VII	110520	40	37	7	1	20.0	0.37	0.20
34,0-38,0	Layer VIII	1558753	60	39	9	1.18	20.0	0.37	0.18
38,0 - ∞	Layer IX	90120	38	37	7	0.98	20.0	0.40	0.22

Where: E: Young Modulus, c': Effective Cohesion, φ': Effective Angle of Internal Friction, ψ: Dilatancy Angle, K0: At rest coefficient of Earth Pressure, K0NC: Normally Consolidated At rest coefficient of Earth Pressure, γ: Total Unit Weight, ν: Poisson Coefficient.

Using the cores of the borings made through drilling, direct correlations can be made between the physical cores and the ID defined by the DMT, this way its established that the silts and sandy silts identified by the DMT belong to Quaternary pyroclastic deposits. The transition with the lahar (pyroclastic mud with high density of rock fragments) was detected between 17.0 and 23.0 meters below ground surface. The groundwater table is located at approximately -17 meters below ground surface.

The typical geotechnical profile at Double Tree and around the area of interest comprises the soil layers in order, as shown in Table 1. It is important to clarify that this is a general soil profile and some of the layers could be absent in some sites.

3 CONSTRUCTION PROCEDURE

Piles for all the projects (Universal, Double Tree, Cosmopolitan and Secrt) were augured cast in place (ACIP). Perforation on the hole was executed using a conventional Kelly bar auger equipment with short barrel for excavation and retrieving of soils. Soil walls were in general stable, so no casing or polymers were used for stabilization. Another tool was required to penetrate to boulders from the lahar layer. Once the hole is excavated to the required depth, and before steel cage installation, a cleaning bucket was used to retrieve sediment from the bottom of the hole.

In Doble Tree, Cosmopolitan and Secrt projects pile steel cages were installed in one section. However, in Universal, due to longer pile lengths, the steel reinforcement cages were installed in more sections, the test pile in three, the bottom with the load cell welded at the middle and two more sections on height that were installed and welded with the previous section once this last was in place. On the test piles this take more time than on other production piles. Although tell-tales and strain gauges were preassembled on each section still was required to run cables and connect tell-tales piles among piles. So, the excavation was opened more than normal i.e. increasing the possibility of additional wall collapses. Furthermore, no other direct cleaning procedures were done after the cage installation.

Concrete on the shaft was placed with a tremie process with the pile starting at 10-20 cm from the bottom. Upper funnel was provisionally covered at the bottom so that it could be filled with concrete and the released to provoke a quick surcharge on the pile toe to allow for displacement of the any remaining sediment. Concrete level increases inside the pile until it is required to elevate the tip of the tremie pile to continue with the flow. At any time tremie should be introduced at least 2-3 meters on the concrete to avoid pile necking. At top, concrete is overflowed to eliminate contaminated concrete. Additional chipping is done after concrete hardening to verify that concrete on the pile top is sound.

4 BIDIRECTIONAL LOAD TESTS

Bidirectional static load tests (BDSLT), were executed on bored piles at Universal Tower (one test) and Double Tree building (two tests). In Universal Tower, (1) BDSLT was performed on a 1.2 m diameter and a 45 m length pile reaching total axial load of up to 22 MN on December 08-09, 2018. In the case of Double Tree, two (2) tests were carried out on 1.0 m diameter piles, (1) 15 m length up to 13 MN total test load executed on March 05th, 2019, and (1) 20 m length up to 16 MN done on March 18th, 2019.

Each BDSLT utilized a hydraulically operated "Cell," with multiple jacks arranged and embedded inside the concrete of the pile before casting (Fig. 2). The cell is installed somewhere along the shaft, on a location intended to equilibrate the soil side friction of the upper section of the pile above the cell with the soil friction and end bearing of the section below it. On Universal this equilibrium point was calculated 4.0 m from the pile toe, meanwhile on Double Tree, the cell on the 15 m pile was installed at 5.4 m from toe and on the 20 m pile at 4.2 m from toe.

Fig. 2. BDSLT testing. Installation of steel cage for the pile with cell welded on. Ref. Photos from Universal Tower project.

5 DYNAMIC LOAD TESTS

Dynamic load tests were performed on piles from two other projects: Cosmopolitan and Secrt sites. One 0.8 m-15 m depth pile was tested at Cosmopolitan on October 16th, 2018, and 2 piles, 1.0 m diameter and 20 m long at Secrt, both on March 14th, 2020. For the dynamic tests, a machined 155 kN free falling weight system was used with a lead that allows an adjustable drop height. Testing piles were prepared with a 1.5 m concrete built-up above the ground so that sensors could be installed, and to provide a flat surface for the weight drop on pile head. A protection steel sleeve was cast with the built-up to avoid head damages from strike.

For each DLT, the following sensors were installed in each pile: (1) accelerometer and (3) deformation sensors at 120° in plane- to account for possible bending effects. Cut rectangles were opened on the protective sleeve for placing the sensors. In Fig. 3, the set-up for the test at one of the sites is shown. Prior to each DLT test execution, a Wave Equation Analysis (WEA) was performed in order to predict the behavior of the pile-weight system after each strike. By means of the WEA analysis, the transferred amount of energy due to the impact was estimated, as well as the induced dynamic stresses on the pile due to the impact and the expected theoretical pile capacity.

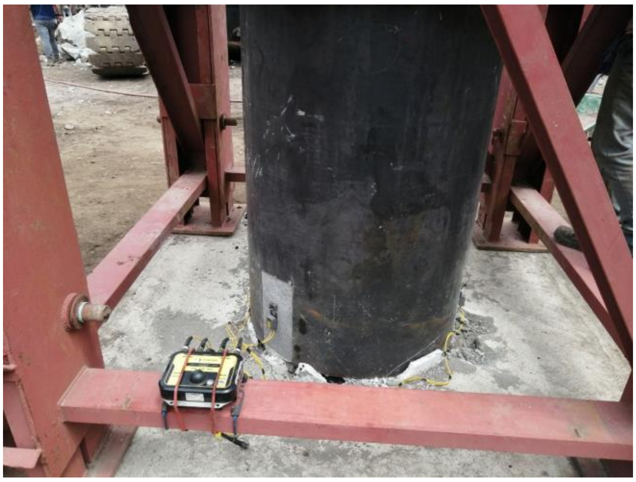

Fig. 3. Set-up for DLT. PDR equipment and sensors on pile, with steel sleeve on head. Ref. Photos from Secrt Tower project.

For the case of bored piles, the diameter is highly variable with depth due to the construction process. In order to draw a plot of diameter versus depth for each test pile, a PIT (Pile Integrity Test) was performed prior to the DLT test. This input allowed an easier and more accurate signal matching process. During the test, the striking mass is lifted and dropped (free fall) from continuing increasing heights. For each drop height, an assessment of dynamic compression and tension stresses is carried out in order to avoid damaging the test pile. After the test, a signal matching process is carried out for a given test pile, with the maximum drop height.

For the Signal matching the measured wave up (WU) was compared with the calculated WU from a WEA model. Static and dynamic parameters of the WEA model were iteratively changed until good agreement between the measured and calculated signals is achieved. The most important result from the calibrated WEA model is the pile vertical capacity, including lateral friction and pile toe distribution of the capacity

6 SINGLE PILE ANALYSIS

In the case of Double Tree, the idea was to analyze and design a combined piled raft foundation (CPRF) by means of 3D FEM MIDAS GTS NX (Geotechnical and Tunneling System). This analysis was carried by Nile Engineering Group form Egypt and M y V Soluciones Geotécnicas from Costa Rica.

As a first phase of the FEM analysis, a single pile analysis (SPA) was executed in order to estimate suitable pile dimensions to be used in the CPRF system at Double Tree Hotel project. The performance of a single pile with different diameters and lengths was analyzed to estimate limiting skin friction and end bearing resistance, which would be applied later for single pile in CPRF analysis by non-linear 3D FEM analysis.

Elastic constitutive model is considered for pile elements, while Modified Mohr-Coulomb Constitutive model is used to represent soil elements, with properties estimated based on the geotechnical exploration (see Table 1). Solid 3D elements are utilized for modeling the pile and surrounding soil elements. Solid to solid interface with Coulomb friction elements were employed for the simulation of load transfer mechanism from pile elements to the surrounding soil elements, through incremental loading steps. The analysis results were used to plot load-settlement curve of the pile and to estimate capacity and examine load transfer mechanism.

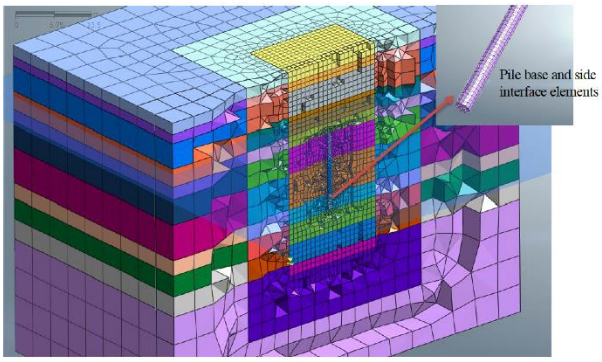

Fig. 4. Single pile analysis and model. Ref. Nile Engineering Group, Egypt and M y V Soluciones Geotécnicas.

Variable elements size was used to allow dense mesh at zones of stress concentration. Enough model boundaries are ensured to avoid any effects of boundaries on pile stresses. Analysis was performed by construction stage analysis, as initial overburden pressure is considered with corresponding zero displacements in elements, followed by stage of excavation, then changing pile volume to concrete elements to simulated casting and, finally, loading phase in which vertical load is applied incrementally and results are saved for future analyses. In Fig. 4, an example of a single pile FEM model is presented, as well as the corresponding contours for radial displacement.

Next, the estimated load settlement curves from SPA for the two type of piles that were selected and implemented in the actual project. Initially the model was run with the geotechnical parameters from Table 1. Later, it was modified to match as much as possible with the behavior of the Equivalent top load curve (ETL) for the respective BDSLT, as obtained in the following section. Fig. 5 shows the case for 1 m diameter and 15 m long pile already adjusted.

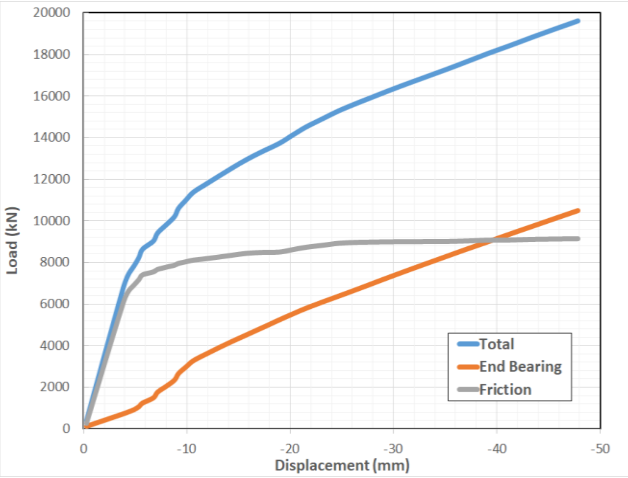

Fig. 5. Load-settlement curves from FEM SPA. L=15 m. Diameter = 1 m. Ref. M y V Soluciones Geotécnicas, Costa Rica.

The matching of the curves from FEM and the load tests, is a very difficult task since there are for this geotechnical profile, 8 parameters and 9 layers i.e. 72 total possible parameters to match. The procedure is changing a parameter, run the software and compare curves from the FEM and the BDSLT-ETL, which is done iteratively. For this case we concentrated on two parameters on the IX Layer, the friction angle (φ') and the dilatancy (ψ), since we thought some of the differences with the curves are related to the upper section of the curves in which the FEM curve need to get stiffer to match the BDSLT-ETL.

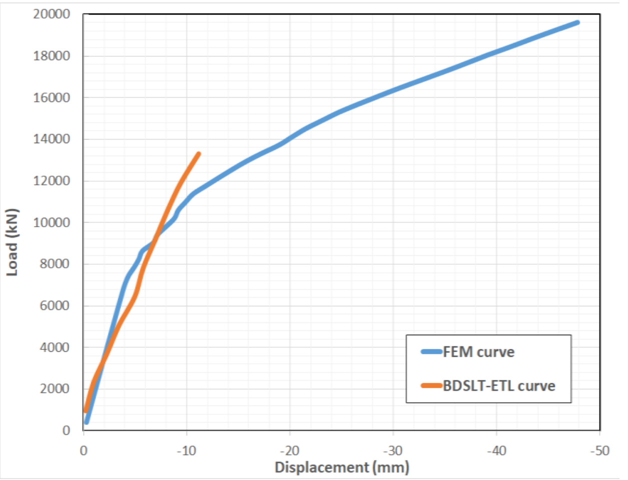

Fig. 6. Comparison of load-settlement curves from FEM SPA and BDSLT-ETL. L=15 meters.Ref. M y V Soluciones Geotécnicas, Costa.

Friction angle was changed from 37 to 42° and the dilatancy from 7 to 12°. The comparison of the adjusted FEM and the corrected BDSLT-ETL is shown on Fig. 6. Although the comparison is good is possible to get a better matching changing other parameters from different layers. The purpose of this article was not to concentrate on this matching process, although the authors recognized this is worth subject that should be further investigated.

7 RESULTS AND DISCUSSION

The typical load-displacement (P-Δ) for a BDSLT, consists of two independent curves, one for the upper and the lower sections of the pile. LVDT and dial gauges show good agreement on tests. The two P-Δ curves can be integrated in an equivalent top load curve (ETL) like the one from a conventional static load test. This can be done by applying the following equation for total load:

$$\boldsymbol{Q_{tot}} = \frac{\boldsymbol{Q_{up} - W}}{\boldsymbol{\gamma}} + \boldsymbol{Q_{down}} \qquad (1)$$

Where Qtot: total load from upper and bottom pile sections, Qup: load from upper pile section, Qdown: load from lower pile section, W: upper pile section weight, γ= 0.9 correction factor.

To determine the displacement for the ETL test, it was considered the elastic compression of pile. For this it was followed the procedure recommended by Kwon et al., which considers the difference of the friction distribution of a BDSLT in which stiffer layers are loaded first, compared to a top-down test where the superficial and more flexible layers are loaded first. Due to this effect and the pile compression, displacements of the ETL test would be bigger than those obtained from the BDSLT. It was concluded that curves from the projects considering elastic compression of pile could be up to the double of the displacement constructing the same ETL without such effect. So, this is important to improve the prediction of the ET curve.

The curves for Double Tree show a very elastic behavior with none or negligible final set. However, this was not the case in Universal, in which the upper section of the pile was also elastic, but the lower section failed, exhibiting continuous penetration under constant load. The interpretation of such behavior is that the pile toe did not contribute to the overall pile capacity. It is plausible that bottom cleaning was insufficient so that pile toe acted against compressible sediment. Under these circumstances, ultimate skin friction for the bottom section of the pile was mobilized.

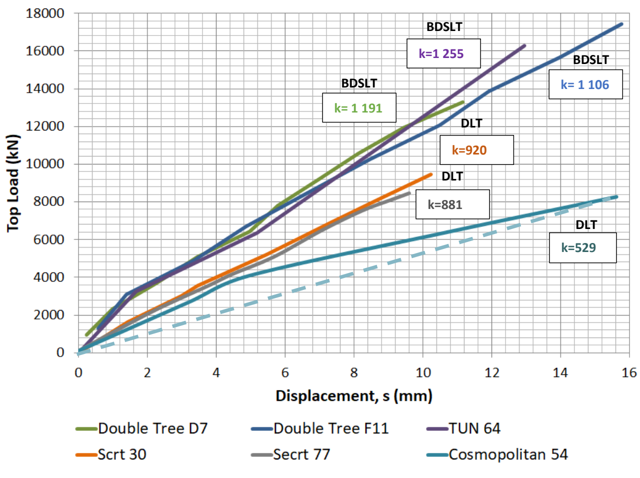

Fig. 7. Load-displacement curves BDSLT-ETL and DLT, calculated unitary stiffness (kN/mm per site Ref. Self-elaboration.

For the case of DLT tests, a load settlement curve can be estimated by utilizing the static components of the calculated total resistance. This can be done after signal matching, and the corresponding curve can be compared with the results of other DLT tests and from BDSLT tests. Fig. 7 shows the load-displacement curves obtained for the BDST and DLT at all the four sites (6 tests in total). BDSLT had been corrected by the elastic compression of the pile, as explained before.

Except for the Cosmopolitan site with an estimated unitary stiffnesses (ratio of load vs displacement) of 529 kN/mm, all the other tests had near values from 881 to 1255 kN/mm, which could be explained because all the sites share a common geology. Once the displacement is corrected to consider the pile compression, BDSLT test matches closer among them, although Double Tree and Universal are not the same site. Unitary stiffness is also similar within the DLT tests. It could be seen that in Cosmopolitan site the first portion of the curve is similar to the curves from the PDA pile tests at Secrt.

Higher stiffness values are mainly related to the surrounding soil strata and how the skin friction is mobilized. Universal and Double Tree have basements so that those piles are located deeper on the profile affecting harder layers. On these same two sites, being the hydraulic load cells location very close to the pile toe, this means that the most competent layers are activated first, compared to a DLT test in which the softer upper layers are initially activated during strike of the mass. This has been addressed through applying the recommendations from Kwon et al to make the BDSLT comparable to a top-down test.

Table 2. Mobilized lateral skin friction per test-project Ref. Self-elaboration.

Project	Tests	Unit mobilized skin friction (kPa)			Displ. (mm)
		Clayey silt	Lahar	Altered tuff	
Universal	1 BDSLT	N.D.	21-22	213-897	104
Double Tree	2 BDSLT	72-86	37-922	N.D.	11-16
Cosmopolitan	1 DLT	30-79	144-215	N.D.	15
Secrt	2 DLT	8-17	N.D.	N.D.	8-10
Max.		**86**	**922**	**897**	
Theorical adherence		**77**	**108**	**502**	

From the strain gauges' measurements, the mobilized skin friction for each pile section could be estimated which relates to a specific soil layer. Table 2 is a summary of the mobilized unit skin friction obtained from the tests at the different project sites. Also, the type of test for each case is pointed out, since friction distribution would be different, as explained. Estimated values are compared with the theorical reference. As can be seen, for the Lahar and altered tuff layers, mobilized unit skin friction obtained from the tests are higher that the theorical unit skin friction. For reference it is established the head top-down displacement related to the skin friction.

8 CONCLUSIONS

Soil profile at La Sabana is composed of a clayey silt overlying a Lahar unit with an altered tuff below the Lahar. Both BDSLT and DLT tests, as well as the FE single pile analysis, help to better understand the behavior of those soil layers, regarding pile foundations, specifically pile capacity and deformation.

On the other hand, high unit skin friction values obtained from the tests show that an optimization of the design and construction is feasible for other bored pile supported buildings and structures around the zone of study. High unit skin friction values obtained from the tests show that an optimization of the design and construction is feasible for other bored pile supported buildings and structures around the zone of study.

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