Dynamic Load Tests at drilled shafts in sandy soil deposits to bedrock, Central Pacific, Costa Rica: strategic learnings for the construction methods.

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Abstract

The foundation of a new building located on the shore of Central Pacific of Costa Rica, a second phase of a complex with existing lower rise buildings, was designed to be supported on 1.2 m diameter short-drilled shafts. The soil profile is a fine sand deposit with high piezometric level due to sea tides, over bedrock. (4) dynamic load tests (DLT) were performed on existing piles not used for the first phase, which have about 2-3 m of rock socket, and new piles with 4 m sockets. However (3) of these tests, (2) existing and (1) new, presented insufficient capacity with null end bearing. Coring trough the centre of these piles and cross-hole logging testing on the new pile, denotated absence of concrete on the bottom. On the other side, the DLT of the last 4 m socket pile, displayed greater than required vertical capacity, with adequate end bearing response. Cross-hole-logging in this pile showed no significant defects. Further calculations with a finite element model (FEM), demonstrates that piles could achieve the required capacity and fixity with a shorter 3 m socket, instead of the designed 4.5 to 5.5 m, representing an important value engineering for the project.

Keywords: drilled shafts, dynamic load test, rock socket, value engineering.

1 Introduction

The project consists of the construction of a building of near 10 stories, that is supported on drilled shafts, on the Central Pacific shore of Costa Rica. This is the second phase of a complex of lower rise buildings.

The design called for short shafts with an overall diameter of 1.2 m and with sockets of 4.5 to 5.5 m into the rock. Dynamic load tests (DLT) as per ASTM D-4945 were proposed to investigate the possibility of reducing the length of pile socket on rock, since it comprises a very difficult execution due the drill will need to penetrate very hard rock.

Additionally, to the DLT, integrity tests were requested, including low strain (PIT) and cross hole logging (CSL), per ASTM D-6760 as part of the quality control for the piles´ construction.

2 General aspects

2.1 Soil investigation

The site had been extensively investigated by Standard Penetration Test (SPT) and flat dilatometer tests (DMT) boreholes.

Specifically, near the new building footprint a DMT sounding shows mainly sand up to the blade refusal (See *Figure 1*). This refusal occurs in rock at about 8.5 m penetration from surface.

The geotechnical company on charge of the investigation, defined the sand characteristics for the sand layers and the rock underneath as per *Table 1*.

Table 1. Sand and rock layers characteristics as per the geotechnical company

Profundidad (m)	ID Capa	E (ton/m²)	c' (ton/m²)	ϕ'	ψ '	ɣ (ton/m³)	K0NC	μ	Esec (ton/m²)	Eur (ton/m²)
0-2,2	Capa 1	2385,36	0,32	41	11	1,70	0,35	0,26	1908,29	5963,41
2,2-3,8	Capa 2	5205,78	0,77	41	11	1,85	0,34	0,26	4164,63	13014,46
3,8-8,5	Capa 3	1381,75	0,00	36	6	1,71	0,42	0,29	1105,40	3454,37
8,5-30	Roca	33884,00	30,00	44	14	2,30	0,31	0,23	27107,20	84710,00

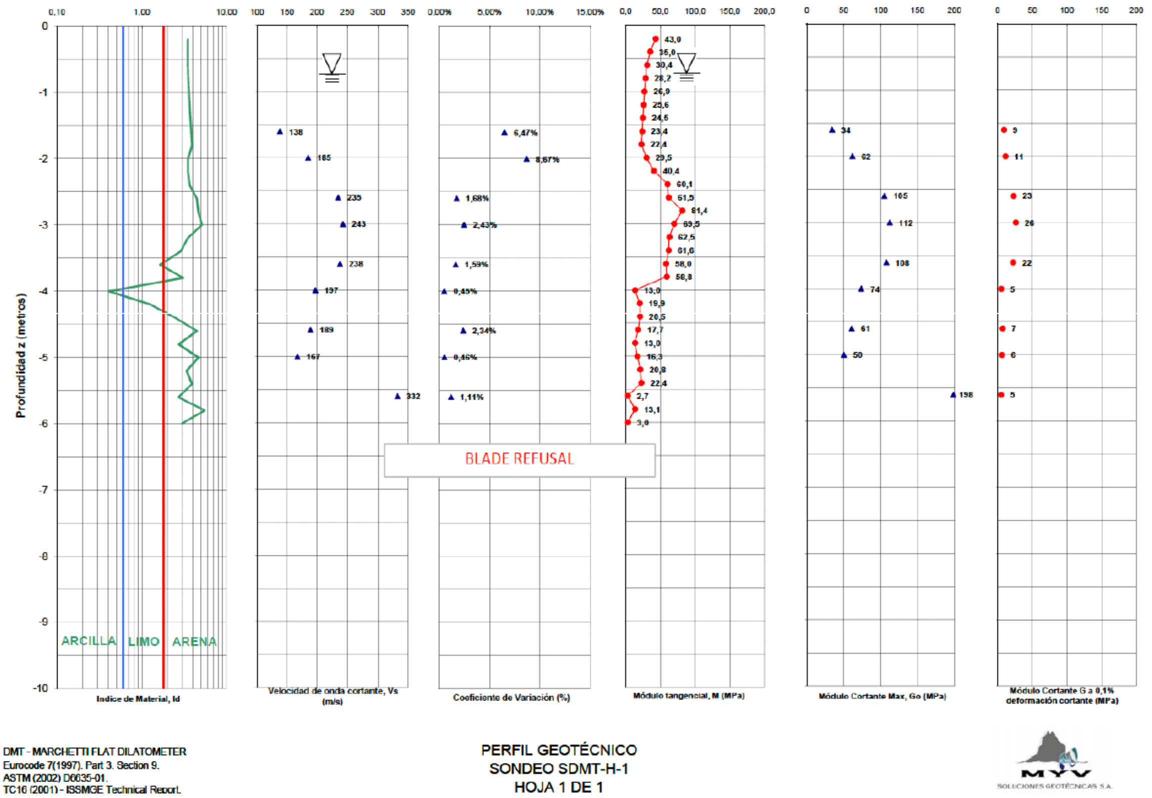

Figure 1. DMT sounding near the building location

2.2 Construction procedure

Piles were constructed by means of a rotatory auger drill with “Kelly” bar. Due to the existence of the loose sand above the rock, and the high phreatic level, a casing was used to maintain the excavation hole and to seal the rock socket.

As indicated later this paper, this was in general unsuccessful, and some sand leakage did occur under the steel case on (3) from (4) testing piles.

For the definitive production piles, the decision was to use polymer slurry in addition to the physical case barrier.

3 Pile testing

3.1 General description

3.1.1 Summarize of the tests

The quality control for the deep foundation (piles) of this project considers dynamic load tests (DLT), as well as integrity tests: cross hole logging (CSL) and low deformation testing (PIT).

Dynamic Load Tests (DLT) were performed, (2) on existing piles constructed for the first phase but not used, with a rock socket of 2.5 m, and (2) in new provisional piles, which have a socket of 4 m.

3.1.2 Pile required capacities

According to the design the factored structural loads for the project piles are as per the second column on *Table 2*.

Table 2. Structural factored and DLT required loads

Type of load	Factored Load (Ton)	Resistance factor	Ultimate test load (Ton)
Usual	835	0.60	1 392
Unusual	942	0.85	1 108
Extreme	1 347	0.95	1 418

This table also shows resistance factors for usual (dead loads), unusual (dead and live loads) and extreme (seismic loads), when DLT testing is used for load confirmation. These resistance factors are taken from the "Código de Cimentaciones de Costa Rica" (Costa Rica Foundations Code).

Ultimate loads for the DLT tests are calculated dividing the factored loads by the resistance factors, which yields the values on the last column of the table. Since all the conditions should be complied with, the minimum load for the DLT tests must be 1 418 Ton which is the maximum from this column.

3.2 Integrity tests

3.2.1 Low strain pile integrity test (PIT)

PITs were executed as part of the quality control, but also to evaluate the location of possible bulges and contractions on the pile diameter.

Figure 2. PIT equipment used for integrity testing

This is required on the wave equation analysis pile model, since the signal matching on the DLT needs a rational distribution of the shaft diameter for accurate calculations. On *Figure 2* it could be seen the equipment used for the PITs.

In this case a double measure was employed, an accelerometer of high sensitivity and a redundant geophone, as shown in the previous figure.

3.2.2 Cross hole logging (CSL)

For the CSL testing it was used the equipment presented on *Figure 3* that includes:

- Central computer and data acquisition (1-3)
- Instrumented pulley to measure depth (5)
- Two hydrophones (6)
- Cables (4,8,9,10)

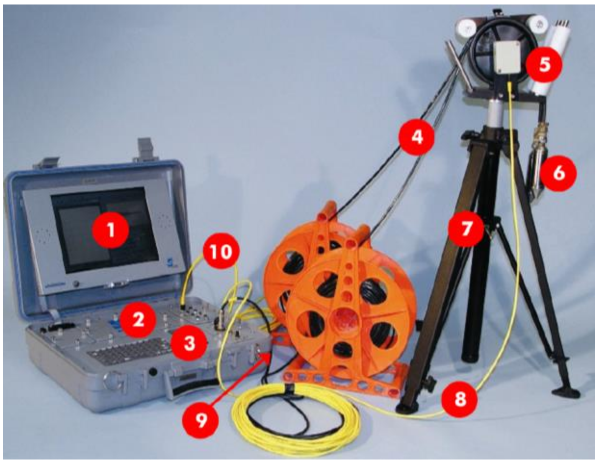

Figure 3. CSL equipment used for integrity testing

3.3 Dynamic Load Testing

3.3.1 Equipment

The DLTs were performed with a pile driving analyser (PDA), that collects the waves information from (3) strain gauges on a 120° pattern to cover the shaft perimeter and (1) accelerometer.

The pile head preparation (Steel plate to avoid concrete spalling and plywood sheet to distribute the blow on the head), the PDA and (2) sensors are shown on *Figure 4*.

As per recommendations, the use of several strain gauges accounts for possible bending effects during the tests meanwhile the acceleration is basically unaffected to this effect, and only one functional accelerometer is needed.

Figure 4. Pile head preparation PDA and sensors

To provoke the high strain wave for the test a fabricated weight of about 20 Ton was used. A lattice crane lift and then free fall the weight onto the pile head.

Figure 5. Weight, guide and crane for DLT

To control the mass fall, a steel guide was placed and centred on the pile to avoid uneven blows and for safety. On *Figure 5* it could be seen the arrangement of the pile, guide and weight used on the tests.

3.3.2 Pre-test Wave Equation analysis (WEA)

Prior to the testing a WEA was done to verify that the required capacities would be achieved during the test, at determinate the weight fall height.

On the other side, due to the normal behaviour of the blow on the pile head, the travelling wave trough the pile shaft is reflected at the toe as a tension wave travelling upwards.

For this reason, principally on a definitive pile, is important to review the tension stresses and compare with the admissible concrete tension resistance to avoid concrete cracking. This also could be done with the initial WEA.

In summary the WEA is usually required to confirm the tested capacity of the pile could be achieved with the weight without exceeding the tension stresses of the concrete.

In this case since the piles are provisional (non-definitive of the project), the aim was to find the higher capacity on compression on the pile, even though the tension stresses become higher than the concrete resistance and damage the pile.

The current analysis results for the project and the fabricated weight for different drop heights are presented on *Table 3*.

Table 3. Results from the WEA

Drop (m)	Displ. (mm)	Energy (kNm)	Stresses (MPa)		Capacity (Ton)
			Comp.	Tens.	
0.5	0.27	40.5	11.02	2.83	1 854
1.0	1.05	81.9	15.76	3.43	2 342
1.5	1.77	123.3	19.30	3.78	2 445
2.0	2.45	164.5	22.23	3.98	2 528
2.5	3.10	205.6	24.79	4.17	2 605

3.3.3 DLT procedure

Once the guide and weight are in position over the test pile head, and the sensors (strain gauge and accelerometer) installed and connected to the PDA, the following parameters of the pile are introduced in the software:

- Diameter (considered constant for the test).
- Modulus of elasticity
- Specific concrete weight
- Wave velocity calculated from above.

The weight is lifted and dropped from different heights starting from a low elevation and increasing it with each blow.

The PDA transforms the strains and acceleration measurements from the sensors into velocity and force. There are several typical behaviors that could be identified from these curves, including:

- Easy or hard driving.
- Toe reaction.
- Whether there is friction resistance or not.
- Integrity condition of the piles.

4 Results and Interpretation

4.1 Integrity tests

4.1.1 PIT low strain testing

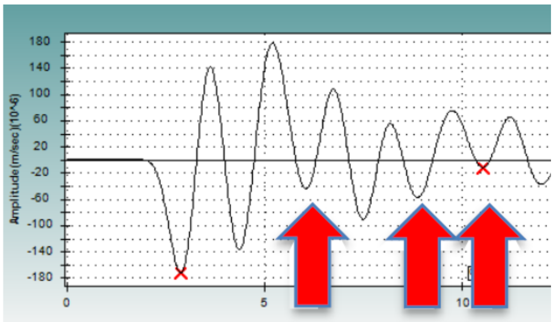

Figure 6. PIT record for one of the test piles

According to the PITs some bulges were detected at different heights. The *Figure 6* shows the record from the test for one of the piles, meanwhile the *Figure 7* how these bulges are included in the pile model as required for the signal matching process.

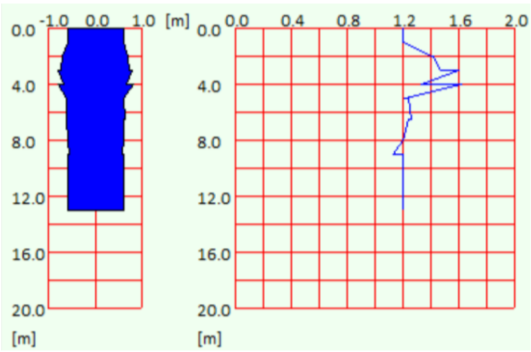

Figure 7. Pile model for the signal matching

4.1.2 Cross hole logging

The (2) existing piles and (1) of the new piles exhibit defects on the toe. *Figure 8* shows 3 sections of the new pile that does not have defects on the toe compared to with *Figure 9*, that presents 3 sections of the other new pile with defects.

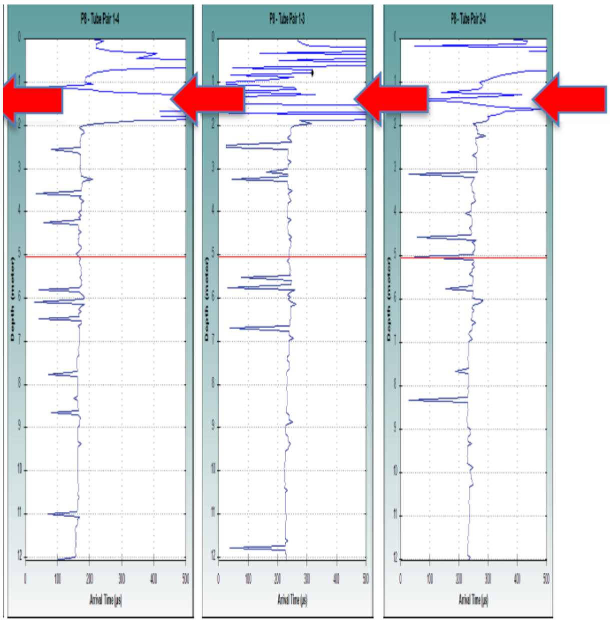

Figure 8. Pile without toe defects

A clarification here the upper section of the piles also shows some contamination, but this is not important for this discussion, since on reality for production piles the concrete on the head would be cut off anyway.

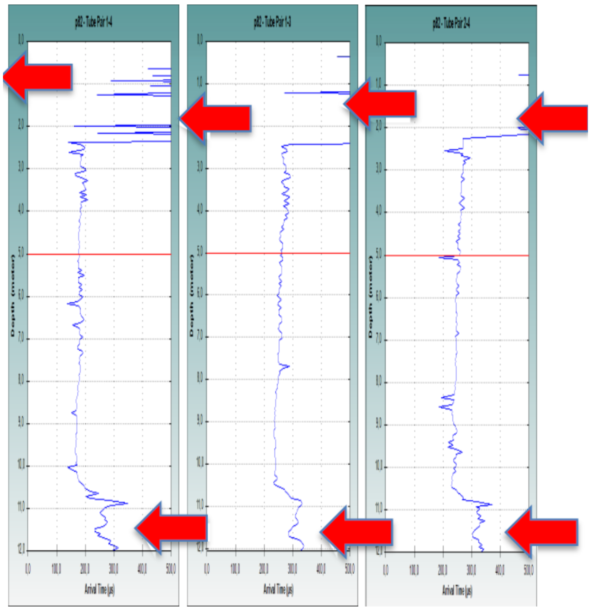

Figure 9. Pile with toe defects

4.2 Dynamic Load Tests

4.2.1 Toe response

The (3) piles in which the CSL denotes a defect on the bottom had a behavior on the wave up (WU) that denote free end (no toe reaction), *Figure 10*.

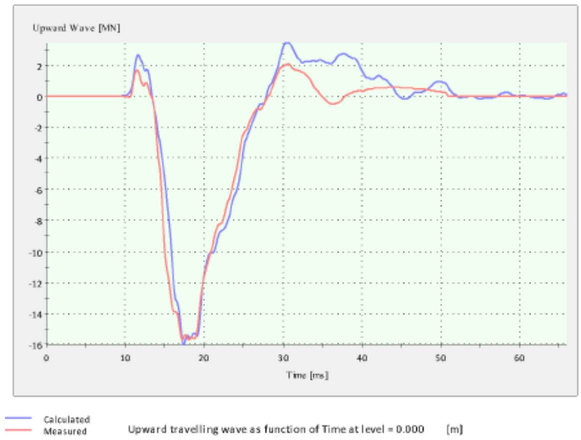

Figure 10. Free end response on WU of test pile

In the mean tine, the new shaft with no defects on the CSL, had a clear response of fixed end, *Figure 11*, which means reaction from the toe.

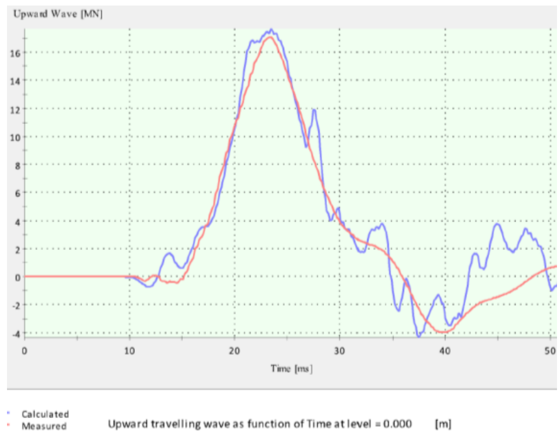

Figure 11. Fixed end response on WU of test pile

Hence, there is a clear correlation between the results from the CSL and the DLT. The difference between free and fixed end is the valley shown on the free-end case, which denotes negative velocity and low force at bottom.

4.2.2 Capacities

As expected, piles with defects and no toe response had low capacities compared with the shaft that do have adequate toe reaction. The results are summarized on *Table 4*.

Table 4. Capacities results - DLT signal matching

Pile	Capacity (Ton)		
	Toe	Shaft	Total
Existing 1	463	217	770
Existing 2	300	421	721
New 1	1 794	666	2 460

It is expected that the friction capacity of these piles is produced principally on the rock socket, with the friction on loose sand meaningless.

Existing and new piles are 1.2 m on diameter, but sockets are respectively, 2.5 m and 4.0 m. A comparison of the unitary skin friction of the existing shafts 1-2 and the New Pile 1 could be done, considering their lateral areas.

This yields values between 23-45 Ton/m^2 for the existing piles and 44 Ton/m^2 for the new pile. Results were not calculated for New Pile 2.

Therefore, the difference between the capacities of the piles with defects and the shaft without them is on the toe capacity itself.

4.3 Drilling

Figure 12. Steel rebar recovered from pile core

Initially, several hypotheses were developed related the behaviour of the shafts with free end, since this is atypical for piles founded on rock:

a. The shaft failed under compression due to bad quality concrete on the bottom.
b. Rock is very fissured near the contact with the concrete shaft, so if failed during the tests.
c. Uncleanliness of the bottom which makes the pile is not reacting against the rock.

To further investigate this behaviour, it was decided to drill and extract (2) concrete cores: (1) through the centre of one of the existing shafts with defects, but also (1) outside the same pile.

From the inside core, there were 0.9 m with neither concrete nor rock recover. Interestingly, the rotation cut some clean steel rebars from the toe, denotating a longer pile length that the obtained from cores (See *Figure 12*).

This "empty space" finished at the elevation where the rock was encountered, which coincides with the nominal pile length. From the outside rotation, the pile socket should be supposedly around 2.5 m, but as indicated was 0.9 m short.

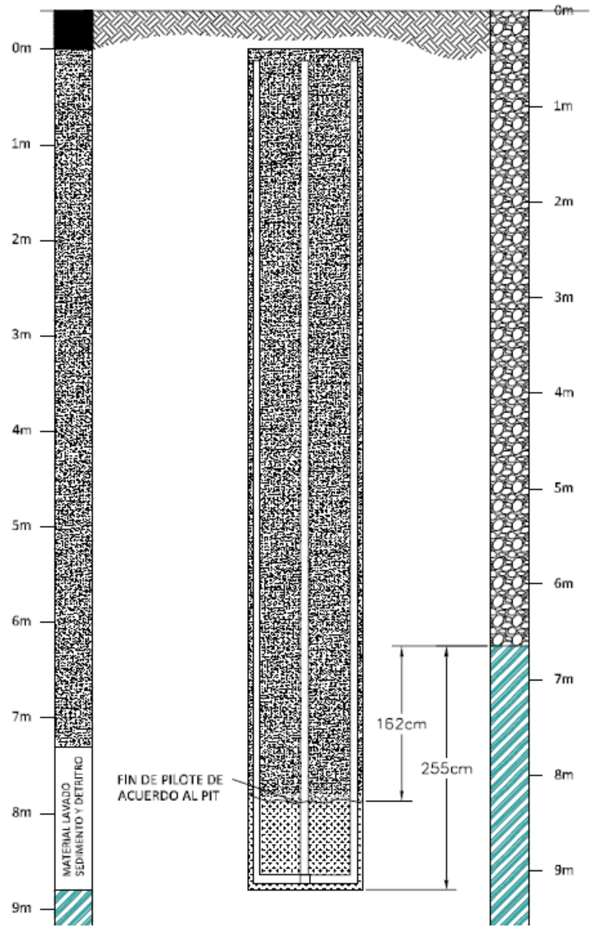

Figure 13. Perforation in and out of existing pile 1

The interpretation of this phenomenon is that from the intended socket the bottom 0.9 m was filled with a combination of rock detritus from perforation and sand fallen into the hole due to insufficient seal. This could be seen schematically on the *Figure 13*.

4.4 Required socket length

Capacity obtained from the DLT on the sound pile, was about 2 460 Ton, which is much higher than the required 1 418 Ton (extremal case, critical). It was then considered to recalculate and optimize the rock socket lengths.

Shaft frictions from the DLT results were used to estimate lateral capacities for a 2 and 3 m socket, and maintaining the same toe capacity, which yields respectively 2 103 and 2 225 Ton.

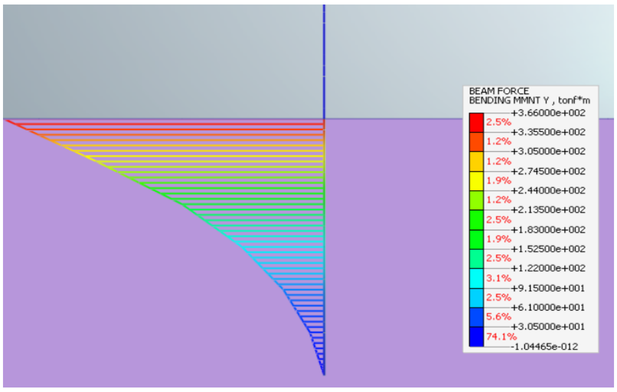

Figure 14. Bending moments for 3 m rock socket

Using the same toe capacity as the DLT test is not necessary conservative, but since both capacities are still higher for several hundreds of tons, this should be acceptable.

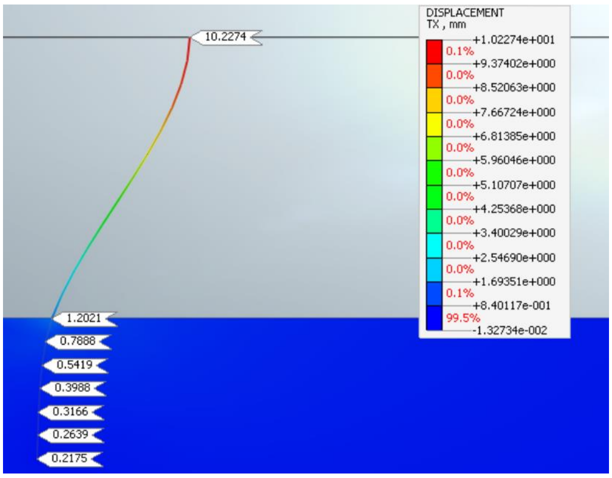

Figure 15. Displacements for 3 m rock socket

Lateral fixity was also investigated, i.e. bending moment dissipation and head deformation under lateral loads. A finite element model (FEM) was used for this purpose (See *Figure 14* and *Figure 15*). The behaviours of a 3 or a 4 m sockets were similar.

Comparison of the maximum bending moments and head displacements for the 3 m and 4 m embedment on rock, are indicated on *Table 5*. Based on these results, was decided that the rock socket could be diminished to 3 m for the construction/production concrete shafts.

Table 5. Moments and displacements from FEM

Description	Units	Rock socket	
		3 m	4 m
Maximum moment	Ton m	366	366
Rock displacement	cm	1.202	1.193
Head displacement	cm	10.223	10.205

5 Conclusions and recommendations

5.1 Conclusions

DLT tests were executed on (2) existing piles with 3 m rock sockets and (2) new constructed piles with 4 m sockets. Quality control was complemented with low strain PIT and CSL tests.

CSL showed defects on the toe on (3) of previous piles, the (2) existing and (1) of the new shafts, but also no significant defects on the other new pile.

This correlates with the toe behaviours and the capacities from the DLTs. Specifically, the shafts with defects had free and response and low overall capacities, compared to the sounded pile which has fixed end response and high capacity.

Core drills inside and outside a shaft with toe defects effectively demonstrated absence of concrete on the tip of those piles. Production piles were produced improving construction methods.

It was also demonstrated that if the piles are constructed properly, they will comply with the capacity requirements. Furthermore, from the results from the DLT test on the sound pile, it was possible by means for a FEM to conclude that 3 m were enough for the rock socket of the new piles instead of 4.5 to 5.5 m.

5.2 Recommendations

There are several recommendations that arouse from the experiences from the case presented in this paper:

a. Quality control on drilled shafts during construction, including integrity tests as CSL and PIT, is mandatory to detect defects that could affect the capacity of the foundations. Integrity tests (CSL) were specified in this project for all the production piles.
b. Additional care should be taken on lithological conditions as in the present case, in which the piles are found on a rock socket, with sand as overburden and high piezometric level. A convenient seal between the case and the soil socket should be good enough to avoid the sand squeeze below the external jacket and contaminate the bottom. Use of perforation slurries could be advisable.
c. The authors recognized that contamination of the pile bottom during construction of concrete shafts is more common than thought on projects in Costa Rica. Many of these cases are undiscovered because limited quality integrity testing is performed for drilled shafts. Moreover, foundations tend to have a length of several meters, and the skin friction, usually higher than theoretical from static calculations, compensates some of the unwanted toe lack of capacity.
d. On difficult soil conditions construction of testing piles could be important to validate and improve the construction procedures. In this sense, it would be always more expensive to apply remedial measures on definitive piles, instead of recognizing issues from the beginning.
e. With capacity testing (dynamic or static load tests) is possible to verify and optimize foundation designs. This is not only because piles are tested on real 1:1 condition, but also because lower safety factors or higher resistance factors are accepted from international and national norms.
f. On site testing in conjunction with computational models (In this case FEM models) can be used to further investigate and optimize the designs for pile foundations.