

Definition and implementation of acceptance process for friction piles driven in sedimentary soils

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ABSTRACT: The extension works of Puerto Bolivar Terminal located in the province of El Oro, Ecuador, include the execution of a deck on piles quay structure. Due to the ground conditions at the project site – characterized by alternate layers of sandy and clayey soils part of the Holocene Quaternary sediments – as well as the maritime environment, the foundation piles are driven steel tubes designed as friction piles. For project design, the theoretical pile bearing capacity was computed in accordance with API standard based on the soil's shear parameters and effective stress values within the ground. This resulted in the definition of target toe levels for the project's piles depending on load demand, pile diameter and stratigraphy. Due to the (easy) driving conditions – no sufficiently competent layer to achieve driving refusal and no significant change in driving parameters between clayey or sandy soils – the end of drive criterion adopted during installation was the target toe level. Dynamic pile tests and associated CAPWAP analyses were carried out during installation at end of driving (EOD) and several days or weeks after installation (Restrike) on the same production piles. This showed a significant – albeit markedly variable – amount of pile set-up, i.e. gradual increase in bearing capacity with time until reaching final / long-term bearing capacity. This paper explains the details of the pile acceptance process and its implementation on site with a focus on risk management. Furthermore, the high set-up factors observed and its impact on friction pile design in sedimentary soils will be discussed.

KEYWORDS: maritime works, driven piles, bearing capacity, set-up, pile testing, dynamic load tests, risk management

1 INTRODUCTION

Puerto Bolivar, part of the municipality of Machala located 150km south of Guayaquil, is one of the world's largest shipment points for bananas with about 80% of Ecuador's production shipped through these port facilities.

In 2016, Turkish port operator YILPORT signed a 50-year concession comprising operation of the existing terminal as well as modernization & extension works, rendered necessary by the significant increase in exported volumes of bananas and shrimps. Phase 1 of the extension comprehends the construction of a 450m long quay and a 12.2 ha container storage yard.

The new quay was designed as a deck on piles structure, the most adequate to resist the important seismic activity in the area – with accelerations at bedrock up to 0.6g and Richter magnitudes higher than 8.

The piles are arranged in 75 bents and 10 rows (A→J), see Figure 1.

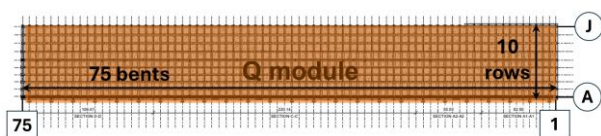


Figure 1. General View.

Due to the geotechnical conditions – sedimentary soils and absence of hard layer – it was evident from the start that the piles would not reach their final bearing capacity during installation (at end of drive).

Thus, the estimation of pile behavior over time and specifically the amount of pile set-up, i.e. gradual increase in bearing capacity with time until reaching final / long-term capacity, was crucial in order to validate the piles.

Historical and more recent works on estimation and theoretical modelling of set-up show a wide range of set-up values along with an important sensitivity to soil type and characteristics. Therefore, a site-specific analysis was indispensable.

2 GEOTECHNICAL CONDITIONS

The project is located in the south-east of the Gulf of Guayaquil, west of the foothills of the western mountain range. The area's geology is characterized by alluvial deposits – resulting from erosion of the nearby mountain range – overlying the Macuchi formation composed of basalts and volcanic sediments.

The alluvial deposits – Holocene Quaternary Sediments – consist of alternate layers of sandy, silty and clayey soils over a thickness of over 140m.

Based on available geotechnical data, comprising boreholes with SPT and piezocone penetration tests, as well as laboratory tests, soil profiles and geotechnical design parameters for the various soil units were defined.

2.1 Soil profiles

The sequence of the different soil layers is fairly the same throughout the project:

- UG2 – Soft Clay, very low strength clay
- UG3 – Sand 0, fine loose sand to silty sand
- UG4 – Clay 1, medium consistency clay
- UG5 – Sand 1, fine medium to dense sand
- UG6 – Clay 2, stiff to very stiff clay
- UG7 – Sand 2, medium-fine dense to very dense sand
- UG8 – Clay 3, very stiff clay
- UG9 – Sand 3, medium-coarse very dense sand

A notable exception being the partial disappearance of soil layer UG7 – Sand 2 towards the north.

To account for variability in thickness and depths of the various soil units, 3 geotechnical profile cross-sections were defined, an example for a geotechnical profile cross-section is given in Figure 2.

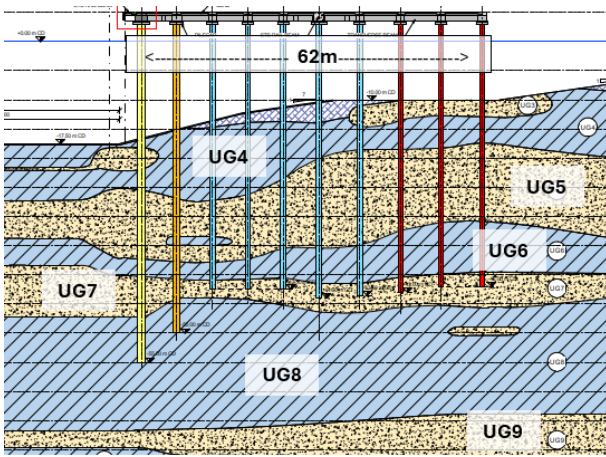


Figure 2. Cross section CC', bents 22 to 57.

Section AA' is applicable to bents 1 to 21 (120m of quay), section CC' is applicable to bents 22 to 57 (230m of quay) and DD' is applicable to bents 58 to 75 (100m of quay).

2.2 Geotechnical parameters

Unit soil weight and design shear parameters were defined taking into account direct test results from laboratory tests and using correlations between in-situ test results (CPTu, SPT – $N_{1(60)}$) and shear strength parameters (drained friction angle Φ' , undrained shear strength s_u), see Table 1.

Table 1. Geotechnical parameters – static conditions.

		UG2	UG3	UG4	UG5
γ_{sat}	kN/m ³	14.5	18.0	17.0	19.5
Φ'	°	-	30	-	35
s_u	kPa	$1 + 1z$	-	$20 + 1.6z$	-
		UG6	UG7	UG8	UG9
γ_{sat}	kN/m ³	18.0	20.0	19.0	20.0
Φ'	°	-	38	-	44
s_u	kPa	120	-	$200 + 2.5z$	-

With z being the depth, in meters, with regard to seabed level.

3 GEOTECHNICAL DESIGN OF THE PILES

Structural design models based on non-linear springs were established using SAP2000 to check the quay's behaviour under static and seismic load conditions, assess internal forces in the structural members and compute vertical loads applied to each pile.

Indeed, one of the major aspects of the verification of a deck on pile structure is the geotechnical capacity of the piles to withstand the applied loads, i.e. the validation of their bearing capacity.

The structural design was optimized in terms of pile diameter (with larger piles being used for longer free lengths) but also in terms of thickness (with smaller thickness being used in depth). The design was also optimized for availability of steel section (which at the time was severely impacted by Covid-related disruption of supply chains). All these factors led to a large number of pile diameters, thicknesses and lengths, which complexified pile design and testing.

3.1 Theoretical bearing capacity

Theoretical bearing capacity, ultimate compressive resistance, was computed in accordance with API RP-2A WSD standard. The required safety factor for static conditions is 2.0 as per the same standard.

For open ended driven steel piles "the total resistance is the sum of the external shaft friction; the end bearing on the pile

wall annulus; the total internal shaft friction or the end bearing of the plug, whichever is less."

$$R_c = R_{b1} + R_{s1} + \text{minimum}(R_{b2}; R_{s2}) \quad (1)$$

With R_c being the ultimate compressive resistance, R_{b1} the wall annulus resistance, R_{s1} the external shaft resistance, R_{b2} the plug end resistance and R_{s2} the internal shaft resistance.

End bearing resistances, R_b , and shaft resistances, R_s , are determined as per equations given in API RP-2A WSD standard. The characteristic shaft friction $q_{s,i}$ is obtained from values of ground parameters as indicated in Equation (2) for cohesive soil – characterized by undrained shear strength s_u – and Equation (3) for granular soil – characterized by an internal friction angle Φ' .

$$q_{s,i} = \alpha \cdot s_u \quad (2)$$

With α a dimensionless factor dependent on undrained shear strength and effective overburden pressure.

$$q_{s,i} = \beta \cdot p'_0 \quad (3)$$

With β a dimensionless factor dependent on relative density and p'_0 the effective overburden pressure.

Prior to design of the project piles, the bearing capacities derived from preliminary pile load tests, detailed in 4.1, were compared to the theoretical long-term capacity computed for soil configuration and characteristics of test piles. The high level of consistency between the results, see Figure 3, further validated the chosen design approach.

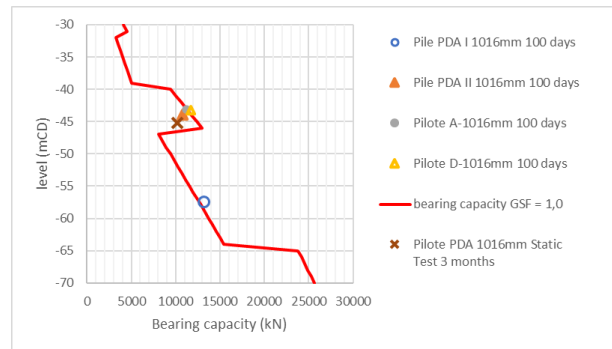


Figure 3. Comparison of theoretical bearing capacity and preliminary test results.

3.2 Design toe levels

Design toe levels of the piles – see results in Figure 4 – were computed conforming API RP-2A WSD standard and considering: soil stratigraphy at pile row deduced from the geotechnical profile cross-sections, geotechnical parameters as per Table 1, pile characteristics (outer diameter and thickness) as defined by the structural design models and maximum static vertical loads applied to each row (A→J).

All pile toe design levels are located in UG7 – Sand 2 layer, except for piles in row A (all bents), piles in row B for bents 22 to 57 – see Figure 2 – and piles in rows H, I & J for bents 58 to 75. For these piles the pile toe is embedded in UG8 – Clay 3 layer.

Also, a parallel assessment pertaining to seismic and post-seismic conditions – in terms of shear parameters and resistances, applied loads and required safety factor – was carried out. The resulting design toe levels were systematically higher, inferring that seismic and post-seismic conditions were not governing the geotechnical design of the piles.

Bents 1 to 12										
Piles / Row -->	A	B	C	D	E	F	G	H	I	J
Φ (mm)	1220	1220	914	914	914	914	813	813	813	813
UPPER THK (mm)	25.8	17.8	19.0	19.0	19.0	19.0	39.0	39.0	39.0	39.0
LOWER THK (mm)	14.7	14.3	14.3	14.3	14.3	14.3	-	-	-	-
TOP LEVEL (m)	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01
CHANGE THK LEVEL (m)	-37	-35	-35	-30	-25	-25	-	-	-	-
TOE LEVEL (m)	-52	-40	-41	-42	-42	-42	-46	-42	-42	-42

Bents 13 to 21										
Piles / Row -->	A	B	C	D	E	F	G	H	I	J
Φ (mm)	1250	1100	914	914	914	914	813	813	813	813
UPPER THK (mm)	22.0	19.0	19.0	19.0	19.0	19.0	39.0	39.0	39.0	39.0
LOWER THK (mm)	14.3	14.3	14.3	14.3	14.3	14.3	-	-	-	-
TOP LEVEL (m)	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01
CHANGE THK LEVEL (m)	-37	-35	-35	-30	-25	-25	-	-	-	-
TOE LEVEL (m)	-52	-41	-41	-42	-42	-42	-46	-42	-42	-42

Bents 22 to 57										
Piles / Row -->	A	B	C	D	E	F	G	H	I	J
Φ (mm)	1250	1100	914	914	914	914	813	813	813	813
UPPER THK (mm)	19.0	19.0	19.0	19.0	19.0	19.0	39.0	39.0	39.0	39.0
LOWER THK (mm)	14.3	14.3	14.3	14.3	14.3	14.3	14.3	14.3	14.3	14.3
TOP LEVEL (m)	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01
CHANGE THK LEVEL (m)	-35	-25	-25	-25	-25	-25	-25	-34	-34	-34
TOE LEVEL (m)	-55	-50	-42.5	-42.5	-42.5	-44	-43.5	-43	-42	-42

Bents 58 to 75										
Piles / Row -->	A	B	C	D	E	F	G	H	I	J
Φ (mm)	1250	1100	914	914	914	914	813	813	813	813
UPPER THK (mm)	19.0	19.0	19.0	19.0	19.0	19.0	39.0	39.0	39.0	39.0
LOWER THK (mm)	14.3	14.3	14.3	14.3	14.3	14.3	14.3	14.3	14.3	14.3
TOP LEVEL (m)	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01	+3.01
CHANGE THK LEVEL (m)	-35	-25	-25	-25	-25	-25	-25	-34	-34	-34
TOE LEVEL (m)	-52	-43.5	-43.5	-43.5	-43.5	-44	-44	-52	-50	-44

Figure 4. Piles' make-up and design toe levels.

For piles whose design toe level is in UG7 layer, an additional verification was carried out to determine the minimum distance from pile toe level to underlying UG8 layer to prevent punching. When found necessary, the toe level was revised.

3.3 Uncertainties and risks

Initially all piles were to be embedded in the much deeper UG9-Sand 3 layer. The detailed design of the applied vertical loads on piles allowed us to optimize the length of the piles.

As can be seen in Figure 5, the theoretical bearing capacity varies significantly at the interface between UG7 and UG8, this is due to the change in limiting behaviour between sandy and clayey soils. Indeed, in sandy soils the limiting factor in Equation (1) is the internal shaft resistance (unplugged behaviour) whereas in clayey soils the limiting factor is the plug end resistance (plugged behaviour).

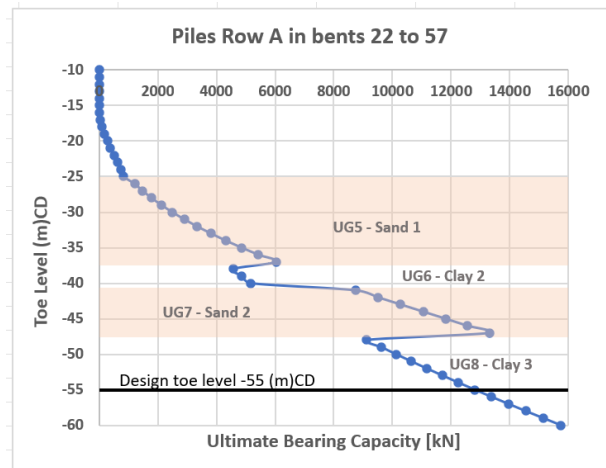


Figure 5. Theoretical ultimate bearing capacity for piles of row A in bents 22 to 57.

This translates as a potential risk of not reaching required bearing capacity depending notably on the actual bottom depth of UG7 at each of the piles' locations whose design toe level is in UG7.

4 DRIVEABILITY ANALYSIS, PILE BEHAVIOUR AND TESTING PROGRAM

4.1 Preliminary tests

Preliminary PDA field tests were undertaken on 6 test piles (OD 1016mm and thickness 20mm) at end-of-drive (EOD) and at several time intervals after installation (restrikes) with one restrike being carried out at ca. 3 months on "pilote de prueba" (test pile). The bearing capacities of the test piles for each PDA test, estimated using CAPWAP analysis, are presented in Figure 6. Additionally, a static load test, performed 2 days prior to the restrike at 3 months on "pilote de prueba", allowed to validate the result obtained through CAPWAP analysis, which was 10000 kN.

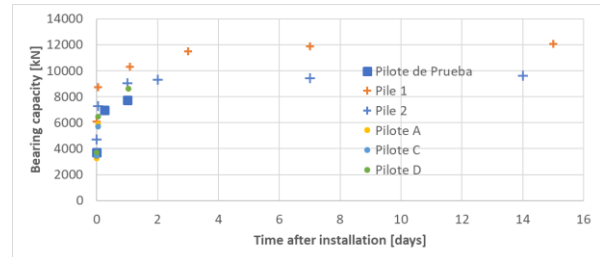


Figure 6. Bearing capacities deduced from PDA tests on preliminary test piles.

As can be concluded from presented test results

1. All piles show a significant set-up over time, the set-up being the factor between EOD and restrike capacity.
2. Long-term set-up factor – at 3 months – for "pilote de prueba" is 2.7.
3. There are significant differences in the evolution of set-up over time.

In the project's geotechnical conditions, the set-up is mainly due to the progressive dissipation of the excess pore pressure in the fine soils, that was produced during pile installation.

Additionally, the analysis of the pile test results showed that generally more than 80% of the bearing capacity is due to friction resistance.

4.2 Driveability analysis

Based on the results of the preliminary PDA tests, Wave Equation Analyses, using GRL WEAP software, were performed to establish site-specific dynamic soil parameters. These site-specific parameters were then used as upper bound parameters. The lower bound parameters correspond to those given by default by the Software. Both of the above mentioned set of parameters were considered to assess the feasibility to drive the piles to their target depth (design toe level) and also to compute the associated expected blow counts at low (50%) and high (85% - 95%) stroke of the hammers available on site (IHC S-150, Junttan HHK 14, IHC S-280 hydraulic hammers).

For the analysis, based on the 2.7 long-term set-up factor found for "pilote de prueba", the soil resistance to driving (SRD) at EOD is taken as 37% of the theoretical long-term axial capacity at target depth.

The results of the driveability analysis show:

- Adequacy of all hammers to install piles to target depth as well as to redrive piles in case of short interruptions
- For restrikes and in case of unplanned longer driving interruptions the IHC S-280 will be sufficient

4.3 Pile behaviour

The expected pile behaviour, considering the geotechnical conditions, the theoretical assessment of the piles' axial

capacity as well as the results of the preliminary pile tests and driveability analysis is as follows:

- Piles will behave as friction piles.
- The driving conditions are easy, notably there is no significant change in the driving parameters (blow count, energy) when passing through the different soil layers or when reaching the target depth. This is also since, for the project's geotechnical conditions and pile diameters, the piles will follow unplugged behaviour during installation (no temporary plug will be formed during driving).
- A significant set-up will take place after installation; thus, at EOD the ultimate long-term axial capacity cannot be directly measured nor will the ultimate axial capacity, as required by design of the structure, be reached.

4.4 Pile testing program during construction

The pile testing plan for the project, established inter alia to comply with AASHTO recommendations, comprises a total of:

- 2 static load tests until failure or 200% of the working load performed on non-working piles. One of these tests was performed on the "pilote de prueba" as part of preliminary testing.
- 3 static load tests until 150% of the working load performed on working piles, each of the tests is in a different geotechnical cross-section, see Figure 7.
- Dynamic load tests (DLTs) performed on over 10% of the production piles, with one PDA at EOD and at least one restrike with PDA.



Figure 7. Location of static load test on working piles (in blue).

The results of the PDA tests performed at EOD are used to establish bearing graphs that allow to estimate the bearing capacity based on blow count and hammer energy for a pile with the same characteristics (diameter and length) installed with the same hammer, an example is given in Figure 9.

5 INITIAL PILE ACCEPTANCE PROCESS

5.1 General Overview

The initially adopted pile acceptance procedure is presented in Figure 8 and detailed below.

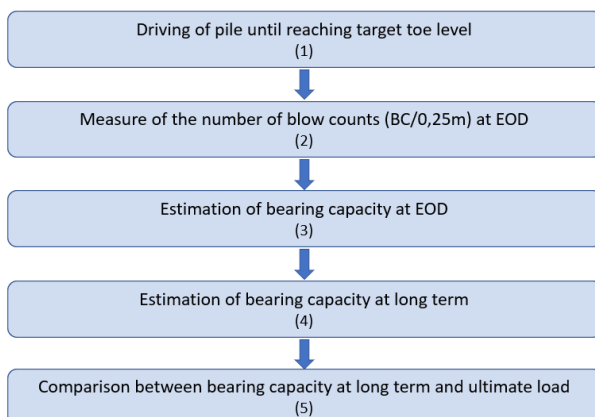


Figure 8. Overview of initially adopted pile acceptance procedure.

1. The end of drive criterion for the piles was defined as the design target depth of the piles, based notably on the considerations detailed in 4.3.

2. Blow counts and corresponding hammer energy (and type) as recorded during pile installation, at EOD.
3. The bearing capacity at EOD is estimated using the bearing graph corresponding to the pile characteristics and hammer used for installation. An example of a bearing graph is given in Figure 9.
4. The bearing capacity at long term is estimated by multiplying the EOD capacity by a set-up factor. This set-up factor is assessed based on the evolution of bearing capacity deduced from various DLTs at EOD and restrikes performed on production piles.
5. If the estimated bearing capacity at long term is equal or superior to the ultimate load applied to the pile, the pile is accepted. If the estimated bearing capacity is inferior to the ultimate load applied to the pile, measures are to be reviewed with the designer in order to ensure sufficient pile capacity.

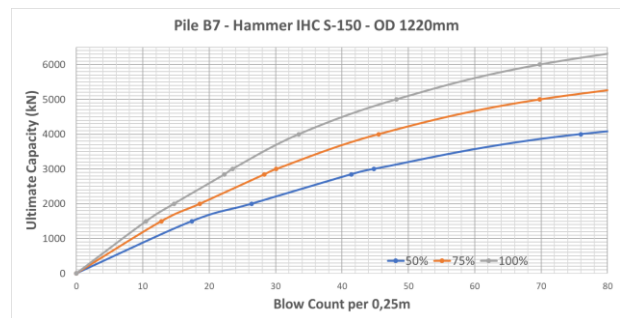


Figure 9. Example for bearing graph – established based on PDA test on Pile B7, installed with IHC S-150 hammer.

5.2 Limitations and uncertainties

Installation of piles started from the south end – bent 1. During the installations of the first production piles, it became clear that the driving parameters (blow count and hammer energy) – and subsequently the estimated axial capacity – at EOD were highly sensitive to several parameters, such as driving speed, installation sequence and type of hammer, some of which are difficult to standardize.

This led to important differences in axial capacity at EOD as estimated based on bearing graphs – differences that were partially corroborated by DLTs performed on selected piles. However, for some piles important discrepancies were observed between bearing-graph-based estimated axial capacity and analyzed axial capacity from CAPWAP results. In case of pile F4 the bearing-graph-based axial capacity estimate was 1815 kN whereas the CAPWAP analysis of the PDA test resulted in an axial capacity at EOD of 2255 kN.

Additionally, the definition of values for the set-up factor has proven to be difficult. In fact, the restrikes performed on the first production piles at various times after installation – ranging from 1 day to 44 days, with some piles being restriking more than once – gave variable results in terms of set-up factor values and evolution of set-up over time, as can be seen in Figure 10.

Rationally the long-term bearing capacity of the piles should not be affected by the driving parameters during installation. Indeed, aside from the momentary increase in overburden pressure due to pile installation with an (impact) hammer, the intrinsic soil parameters are not affected at a scale impacting axial capacity. Therefore, the variation in set-up factor for similar geotechnical conditions – as is the case for this project – is seemingly linked to the differences in EOD axial capacity as well as slight variations in presence of fine soils along the quay.

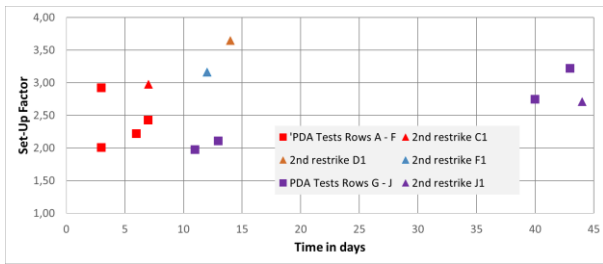


Figure 10. Evolution of set-up over time based on restrike results of the first production piles.

Given the above detailed limitations and uncertainties, the initial approach was firstly modified by adding short term and long term restrikes in order to establish trend curves – showing increase of axial capacity over time – and validate ultimate bearing capacities. Furthermore, a minimum value of capacity to be reached at EOD (25% of ultimate load applied to pile) was introduced and contingency measures if this criterion was not met defined (additional short term restrike and comparison with trend curve).

However, the initial and the modified approach were not deemed sufficiently robust to demonstrate that all the piles will achieve their required axial capacity, corresponding to the ultimate load applied.

6 REFERENCE PILE APPROACH

In order to provide the client with the assurance that a driven pile would achieve the required bearing capacity, the reference pile approach was developed jointly with the designer and the client. It includes several concepts of the initial and the modified approach, such as EOD axial capacities estimated based on bearing graphs and trend curves based on short and long term restrikes on piles.

6.1 General overview

The reference pile approach is presented in Figure 11. The associated glossary is:

- RBC = required bearing capacity. Corresponds to the maximum ultimate load applied to the pile resulting from the structural design.

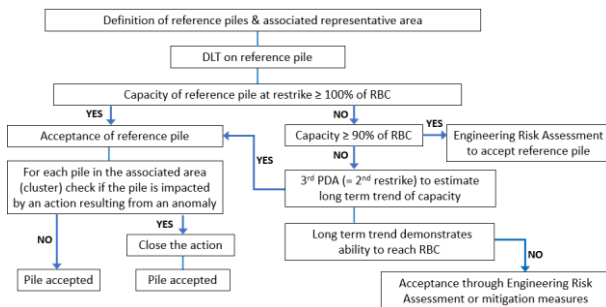


Figure 11. Overview of reference pile approach.

As can be seen, the cornerstone of this approach is the definition of representative piles and their associated representative areas. This allows for a more localized definition and follow-up of pile behaviour in terms of axial capacity.

Reference piles have an identical diameter and comparable design toe level as the piles in the associated representative area. The representative areas, also clusters, were defined considering an influence radius of ca. 20m, as the ground conditions can be considered as nearly identical. – in addition to the overall low variability of ground conditions through the project –

For practical purposes, the RBC to be demonstrated on the reference pile corresponds to the most heavily loaded pile in the cluster.

As an example, the reference piles (blues squares) and associated representative areas (limited by dotted red lines) for bents 58 to 75 are presented in Figure 12.

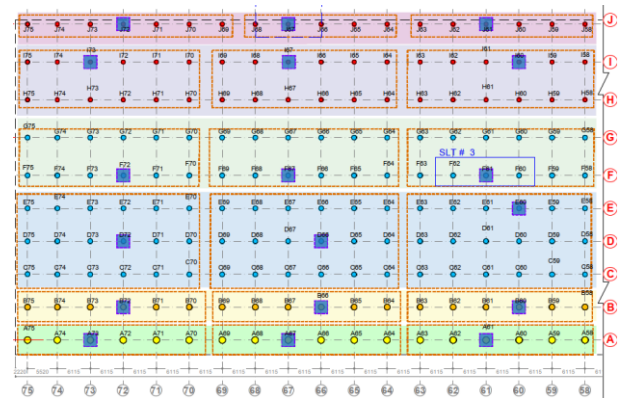


Figure 12. Reference piles and representative areas in bents 58 to 75.

For each reference pile a corresponding bearing graph is established based on the PDA test at EOD. Also, whenever a different hammer from the one used to drive the reference pile is utilized in a cluster, an additional PDA test at EOD is performed to establish the bearing graph for the cluster specific to the alternative hammer.

6.2 Acceptance process

Acceptance criteria are defined based on the axial capacity obtained at restrike on the reference pile (CAPWAP analysis of PDA test results):

- If the capacity at restrike is $\geq 100\%$ of RBC, all piles located in the cluster are considered to achieve required capacity – except if an anomaly is detected (refer to 6.3).
- If the capacity at restrike is $\geq 90\%$ of RBC and $\leq 100\%$ of RBC, an engineering risk assessment (ERA) is required to validate the capacity of the reference pile and by extension all piles located in the cluster – except if an anomaly is detected (refer to 6.3).
- If the capacity at restrike is $\leq 90\%$ of RBC, then a second restrike will be performed on the reference pile to supply additional data for establishing the long-term trend of the evolution of axial capacity.

Trend curves are specific to a geotechnical cross-section profile (AA', CC', DD') as well comparable length, meaning that only restrikes of piles located in the same rows as included in the representative area, will be considered to assess the long-term trend. An example for a trend curve is given in Figure 13. The tendency curve was defined using a normalized set-up curve as proposed by Tan (2004).

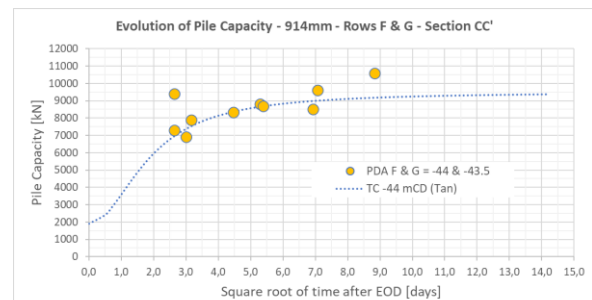


Figure 13. Trend curve for piles located in rows F & G, Section CC'.

The comparison between trend curve and RBC is presented in an ERA.

- If the trend curve demonstrates the ability of the reference pile to reach the RBC, then the reference and all piles located in the cluster are validated – except if an anomaly is detected (refer to 6.3).
- If the trend curve does not demonstrate the ability of the reference pile to reach the RBC, further ERA is done to either accept the piles or define mitigation measures.

6.3 Anomaly: Definition and resolution

Possible anomalies – or anormal behaviour of piles – are detected during installation (initial drive) via the monitoring of blow count and associated hammer energy.

An anomaly is defined by either:

- A bearing-graph-based estimation of EOD capacity of the pile more than 12% below the average value of EOD capacity found for all the piles of the cluster.
- Strange behaviour detected during driving in terms of blow count and associated energy.

The resolution of an anomaly requires an ERA, which assesses whether the anomaly constitutes a risk for pile to not reach required bearing capacity. If a risk is identified the ERA defines an action plan to mitigate the risk.

6.4 Engineering risk assessments and risk management

All data relevant to pile driving and testing (driving logs, CAPWAP analysis results, ...) were transmitted to the design offices to

1. Analyze the driving logs to detect abnormal behaviour during installation.
2. Compute the average bearing-graph-based EOD capacity for all piles of the cluster driven with the same hammer, to identify possible anomalies.
3. Define RBC value for the cluster and compare it to restrike capacity of the reference pile.

If required, an ERA was established. For example, in case an anomaly was detected, the modus operandi was to compare the default in % compared to average with % of RBC on pile compared to RBC for cluster. If this was “insufficient” then trend curves were used to validate ultimate capacity.

The continuous coordination and direct and clear communication between design and site was the most important factor for efficient and swift risk management. Also, for some items, i.e. bearing-graph-based EOD capacity estimates, a double check was performed by design and site, which further reinforced the robustness of the process.

7 CONCLUSION

The Puerto Bolivar project was atypical for a number of reasons. In terms of driven pile behaviour, the project provided valuable lessons, which are detailed hereafter.

For specific configurations, as encountered in this project, the set-up phenomena is primordial to ensure that piles reach the required bearing capacity in the long term.

It is important to be aware of the difficulties associated to “easy driving conditions”: in multilayered sedimentary soils without significant change in terms of driving behaviour, the long-term bearing capacity cannot be directly measured or estimated at end of drive. This constitutes a significant challenge as, at end of drive, an uncertainty exists that the pile will reach the required bearing capacity. This is contrary to most projects involving driven piles, as usually the piles are driven to refusal or quasi refusal (end of drive criterion related to blow count and energy) meaning that they reach the required

capacity at end of drive and the set-up factor is close to 1.0 (coarse sands).

Set-up phenomena (set-up factors and evolution with time) are very site specific, depending on ground conditions and installation methods and sequence. Therefore, it is important to devise a test program that allows to define long-term (or final) bearing capacity for all relevant configurations as well as the evolution of bearing capacity over time. Indeed, this data is essential to remove the uncertainty and allow piles’ acceptance.

For the Puerto Bolivar project, in addition to the total of 5 static load tests performed on working (3) and non-working (2) piles, more than 12% of the 750 working piles were dynamically tested – exceeding the AASHTO recommendations.

This extensive testing, including several (very) long-term restrikes, some of which were carried out more than 200 days after pile installation, was capital to gain insight into the particularities of the site-specific set-up phenomena characterized by the important set-up factors in the range of 3 to 4, with several very long-term values notably higher than 4. These values differ substantially from the usual values of 2 to maximum 3, that are found in specialized literature.

Another relevant finding was the relatively long time needed to reach the required bearing capacity (on average 30-50 days). This complicated the construction process of the deck, as the civil works had to wait for the piles’ acceptance, which was based on the results of the restrike DLTs.

The project’s complex validation process generated heavy interfaces, thus requiring quick and clear communication and rapid completion of the analysis by all actors involved.

All is well that ends well: Applying the pile acceptance procedure detailed in this article and thanks to the efficiency of all participants, all piles were validated, and the project was transferred to the client on the agreed date.

In light of above detailed findings, we would recommend for future projects in similar conditions to schedule a complete testing program – prior and during installation – in addition to theoretical calculations and assessments, in order to define site-specific pile behavior and set-up.

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