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Pile bearing capacity of the new bridge over Zambezi River (Mozambique): Predictions and performance of static load test results

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ABSTRACT: The evaluation of pile bearing capacity is dependent on the interpretation of the pile head load-settlement curve and the determination of the shaft and toe resistances in careful and well instrumented tests. It is then possible to calibrate design methods with load transfer curves at different depths for progressive loading levels. In the New Zambezi Bridge (Caia – Chimuara, Mozambique) deep foundations have been designed taking into consideration different methodologies: methods based directly in SPT data (Reese & O'Neill) and methods based in CPT data (Bustamante & Frank, 1999), giving rise to some scatter in final results. For the complete and trustful solution, static load tests on some piles were executed using the Osterberg test method, in order to obtain a simple way to evaluate shaft load transfer and define the applicability of each empirical methodology. Results of this comparative analysis are proposed and some suggestions are presented for regional practice.

1. INTRODUCTION

Evaluation of bearing capacity of piles (base and side friction) as well as the associated settlements for its full mobilization is a matter of great discussion in foundation engineering. Pile load testing is an important tool to calibrate and validate design models for any specific situation, as well as to achieve a better understanding of its adequacy. In this work, the data related to the foundation design of the new Zambezi River Bridge (Mozambique) is presented and discussed, based in the results of Osterberg Pile Testing.

2. GEOLOGY

The geology in the study area is fully described in the 1:1.000.000 geologic map "Moçambique", by *Instituto Nacional de Geologia do Ministério dos Recursos Minerais.* Alongside with the above mentioned geological map several other documents were also taken into consideartion.

From the geomorphologic point of view, the Zambezi river vale is represented by several flat surfaces associated to 4 main erosion cycles, as described below:

a) ***Mountain Zone, associated to the Gonduana cycle.***
b) ***Great Plateaus region, associated to the African cycle.***
c) ***Medium Plateaus associated to the Zumbo cycle.***
d) ***Coast Plateaus associated to the Congo cycle (Chire and Zambeze Plateaus)***

The site is located within the last geomorphologic unit, which contains some tectonic aspects, important for the area structural modelling. The Chire and Zambeze Plateaus are underlain by ***Quaternary*** formations of alluvium nature, showing a very significant extension, developing from the south border of Malawi to the Zambeze River. The Zambezi valley in the site area corresponds to a "Graben" type structure. As a result, it presents as a stepped shape topography, perfectly visible in the topographical map of the study area (the valey zone corresponds to the lower plate and the margins correspond to the elevated zone). The Zambezi River type of delta shows that fluvial action prevails towards the marine action. On the other side, the alluvium plateau of the Chire River lays on a portion of the Niassa Rift, designated by Chire Graben. This particular area has presented an intense seismic scenario in the past.

From the lithologic point of view, the site underlying regional geology associated to the Zambezi river valey, displays *Quaternary* (sands, silts and clays of old alluvium deposits), *Cenozoic* (Sandstones of Inhaminga and Chicolane) *and Pre-Cambrian* (Gneisses) formations.

3. GEOTECHNICAL SURVEY

The geotechnical characterization campaign was based in boreholes with SPT and Cross-hole seismic testing, as well as laboratory testing. A summary of the obtained results are presented in Tables 1 to 5.

Table 1. SPT testing

Horizon	Litology	N_{SPT}
A	Earthfills / organic soils	< 10
B	Clay and silty deposits	8 - 60
C	Sandy deposits	10 - 60
D	Sandstones	> 60*
E	Gneisses	> 60*
* - penetration < 15cm		

Table 2. Soil identification

Type of soil	#200 sieve (%)	Clay fraction (%)	Plasticity index (%)
Sandy (17,1%)	< 35	< 15	NP
Silty (34,3%)	60 – 100	5 – 90	< 15
Clayey (48,6%)	> 95	40 - 85	30 - 55

Table 3. Physical characterization

Type o soil	Water content (%)	Sat. degree (%)	Void ratio	Unified class.
Sandy (17,1)	15 - 20	> 75	0,5–0,8	SM SP
Silty(34,3%)	20 - 30	> 90	0,5–0,8	ML MH
Clayey (48,6%)	15 - 30	> 90	0,5–0,8	CH CL

Table 4. Cross-hole seismic testing (sands)

Sands		
N_{SPT}	Vs (m/s)	G_0 (MPa)
1 - 5	50 - 100	8 - 35
5 - 10	100 - 150	35 - 75
10 - 30	150 - 275	75 - 115
30 - 60	275 - 400	115 - 150
> 60 (1)	400 - 575	150 - 180
> 60 (2)	> 575	180 - 350

Table 5. Cross-hole seismic testing (clays)

Clays		
N_{SPT}	Vs (m/s)	G_0 (MPa)
0 - 4	80 - 140	10 - 30
4 - 8	140 - 180	30 - 55
8 - 15	180 - 230	55 - 95
15 - 30	230 - 300	95 - 175
30 - 60	300 - 400	175 - 320
> 60 (1)	400 - 520	320 - 590
> 60 (2)	> 520	> 590

4. METHODOLOGY

The evaluation of bearing capacity of piles was performed following the LCPC methodology (Bustamante & Frank, 1999), based on extensive full scale pile load test data from France and supported by pile load test data from North America (Reese & O´Neill, 1999). In this sense the bearing capacity of an isolated pile can be calculated using the following expressions:

$$R_c = R_s + R_b$$
$$R_s = q_s\, A_s$$
$$R_b = q_b\, A_b$$

where R_c represents the bearing capacity, R_s shaft capacity, R_b the toe bearing capacity, q_s e q_b respectively the net side friction and toe bearing resistances and A_s e A_b the lateral and toe areas. The general safety factors recommended (DTU, French code of Practice, 1999) are 2 and 3 for R_s e R_b, respectively:

$$R_d = (R_s/2) + (R_b/3)$$

The evaluation of toe and side friction resistance (q_s e q_b) was based in the methods proposed by the Federal Highway Administration (Reese & O'Neill, 1999) and by French Code of Practice (Bustamante & Frank, 1999).

The first method considers that stress relief and excavation disturbance lead to a reduction of the friction angle to critical state (constant volume) values. Therefore, side friction is mainly dependent on horizontal stresses, and can be written as follows:

$$q_s = \int_0^L K\, \sigma'_z \tan\phi_{cv} = \int_0^L \beta\, \sigma'_z$$
$$\beta = 1{,}5 - 0{,}135\, \sqrt{z}$$

where z represents depth, K the radial earth pressure coefficient, σ'_z the vertical effective stress and ϕ_{cv} the friction angle at constant volume.

According to the same methodology, considering the sandstones as a non-cohesive intermediate geomaterial (IGM) toe bearing can be evaluated by the following expression:

$$q_b = 0{,}59\, [(N_{1(60)})]^{0,8}\, \sigma'_{ZD}$$

where $N_{1(60)}$ is the corrected SPT value and $\boldsymbol{\sigma'_{ZD}}$ the effective stress at foundation depth. Toe bearing is determined considering in the infkuence zone 8 diameters above and 3,5 below the pile tip. The level of mobilization for toe bearing determination is taken as 5% of pile diameter.

Considering LCPC methodology, the net toe-bearing and side friction resistances, q_b e q_s, can be estimated by the following expressions:

$$q_b = K_c \, q_{ca}$$
$$q_s = (1/\beta) \, q_c$$

where q_c is the cone penetration resistance from CPT, q_{ca} equivalent cone penetration resistance in a zone 1,5D above a below the tip, K_c a capacity factor dependent of the soil type, compaction level and pile type (Group I-0,15, in the present case) and β a side friction coefficient depending on the same variables (Grupo I A - 60 to 150). The considered conversion factors for $N_{SPT} - q_c$ (CPT) ratios were within 0,1 (clays) to 0,4 (sands) interval.

The mobilization of both toe bearing and side friction are strictly dependent on the level of settlement. Generally the total mobilization of side friction occurs for very small settlements (10-15mm), while toe bearing only reaches its maximum value with settlements of about 10% the diameter of driven piles and 40% of diameter of bored piles (Fioravante, 1995).

The load settlement evaluation was based upon Fellenius (1999) method which can be expressed simply by the following equations:

$$(q'_b)_m / q'_b = (\delta/\delta_u)^g$$

$$(q_s)_m / q_s = (\delta/\delta_u)^h < 1$$

where $(q'_b)_m$ is the mobilized toe resistance, q'_b the net toe resistance, q_s the unit side friction, $(q_s)_m$ the mobilized side friction, δ the actual settlement, δ_u the required settlement to fully mobilize the ultimate bearing capacity (10mm for side friction and 10% of diameter for toe bearing), g a exponential factor for toe bearing (0,5 for clay and 1,0 for sand) and h is the side friction exponential factor (0,02 to 0,5 for clays to sands, respectively)

In context of side friction mobilization, there are some different approaches to estimate the maximum settlement, which can be divided as follows (ABMS/ABEF, 1998):

a) Full mobilization dependent on pile diameter which will be 0,5 to 2% for clays and 1 to 3% for sands
b) Full mobilization independent of *pile type and dimensions* and *type of soil*, where the reference value is 10mm, although some authors have been trying to define a wider range of values (ex. Jamiolkowski & Lancellota, 1988)

The methodology used in calculations (Fellenius, 1999) pointed out 10mm for side friction mobilization, being the adopted value for design purposes. On the other hand, the ***h*** exponent parameter to input in the basic formulae ranges from 0,02 to 0,5, from clays to sands (Fellenius, 1999). Since the geomaterials existing at the foundation level are intermediate, well-grading materials, composed by sand, silt and clay, a value of 0,3 was considered.

5. LOAD TESTS - DATA ANALYSIS

In order to calibrate the design modelling a Static Load Test (Osterberg type) was performed in a reference pile (PV14), with 59,5m depth (5,19m from the toe to the load cell) with 3 strain gauges (Geokon), located in 3 main levels: 59,2, 32,2 e 10,1m. These locations were chosen to match with the main geological features represented in the referenced bore-hole, which indicates a medium compact sandy layer (10,5m depth), followed by a firm to hard clay unit (up to 32m) and another compact to very compact sandy layer (up to 55,5 m depth). All these layers stand over the sandstone units. During the test a maximum load of 10,9 MN was applied followed by unloading.

5.1. Toe Bearing

Toe bearing capacity analysis revealed that for a maximum applied load of 5315 kN, a settlement of 30mm (2% pile diameter) was mobilized, clearly within the elastic behaviour. Toe bearing predictions showed 8143kN and 7700kN, following LCPC (Bustamante & Frank, 1999) and Reese & O'Neill's (1999) methodologies, respectively. For these calculations a N_{SPT} value of 120 was taken, since in sandstones the limit of 60 blows was reached for penetrations smaller than 15cm. Even considering the different levels of settlements between test (2%) and predictions (5%) the convergence between them is notorious, as it can be observed in Figure 1. Moreover, using Reese & O´Neill diagram (1999) a value of 8000kN can be extrapolated for 5% settlement, confirming the previous assumptions.

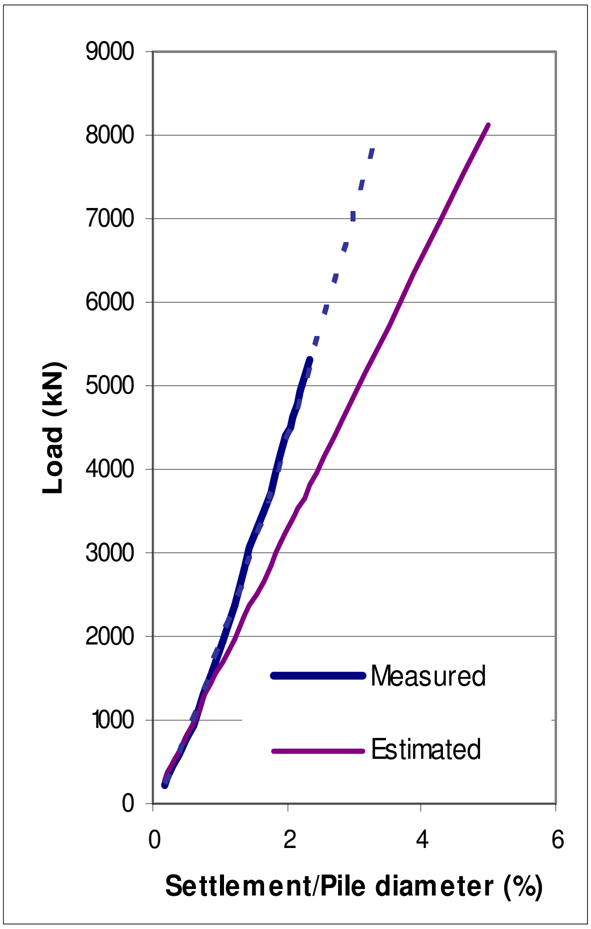

Fig. 1 Toe Bearing:Estimated and measured values

On the other hand, toe bearing exponential factor (g) for settlement analysis (Fellenius, 1999) reflects well the load test results showing a best fit value of 1,05, quite close to the recommended one (1,0).

5.2. Side Friction

The overall side friction test results are presented in Table 6. In Figure 2 the obtained side friction results are compaired with those estimated by the two design adopted methodologies. In Figure 3, side friction transfer loads determined from strain gauges related to each layer are presented while in Figure 4 the mean value is represented and compaired with those estimated by Fellenius method (1999).

Table 6. Load Test results

Strain Gage	Depth (m)	Side Friction (kPa)	Side Friction (kN)	Total Side Friction (kN)
3	10.00	5	235.6	235.6
2	32.15	22	3340.1	3575.8
1	59.16	58	7420.6	10760.7

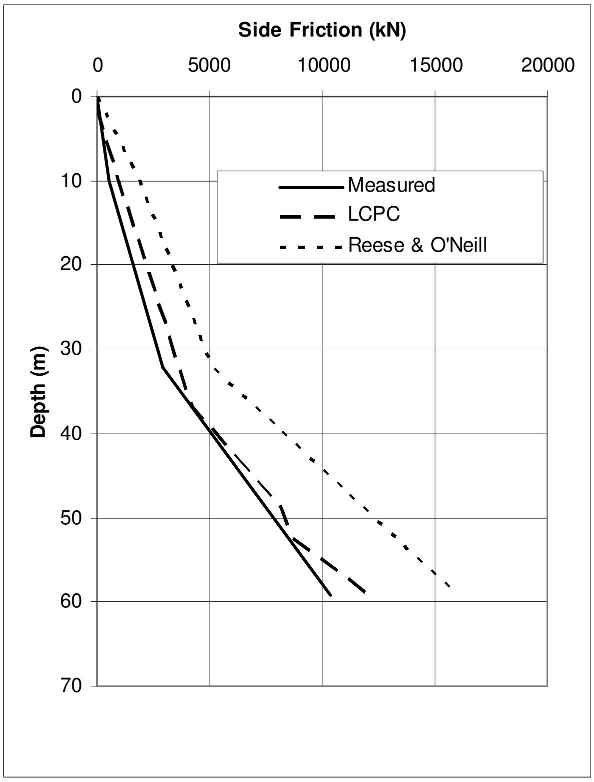

Fig. 2 Side Friction:Estimated and measured values

As it can be seen, the two methodologies somehow overestimated results, considering the reference test. This may be related to the silty fraction present in the sandy soils which should imply an overestimation of the friction angle. On the other hand, the mobilization of side friction as function of the pile movement was also studied and compared with the estimated settlements. For this situation, the load test gives 11,0mm (0.75% of the diameter), within the expectable results considering both the independent approach (10-15mm for total side friction mobilization) or the one dependent of pile diameters and type of soil.

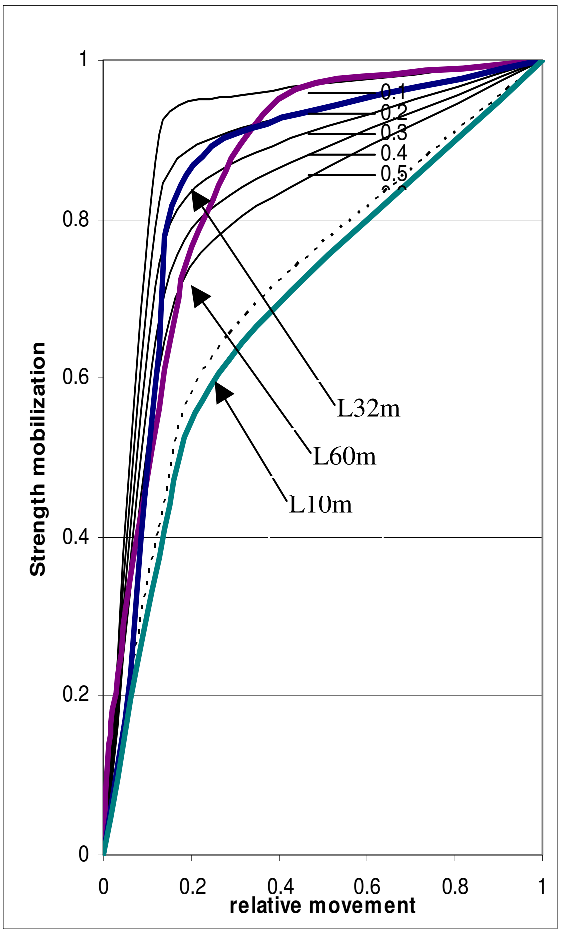

Fig. 3 Side Friction mobilization versus movement (layer by layer)

In Figure 3 the results are represented by the individual strain gauges reveals that the exponent *h* best fit for the first layer is very high (0,9) when compaired with the reference range (0,02 to 0,5). The other two adjusted curves are within the expected ranges, indicating values of *h* of 0,1 for silty clays and 0,2 for silty fine sand. On the other hand, Figure 4 shows that best fit for the mean value would be somewhere around 0,3.

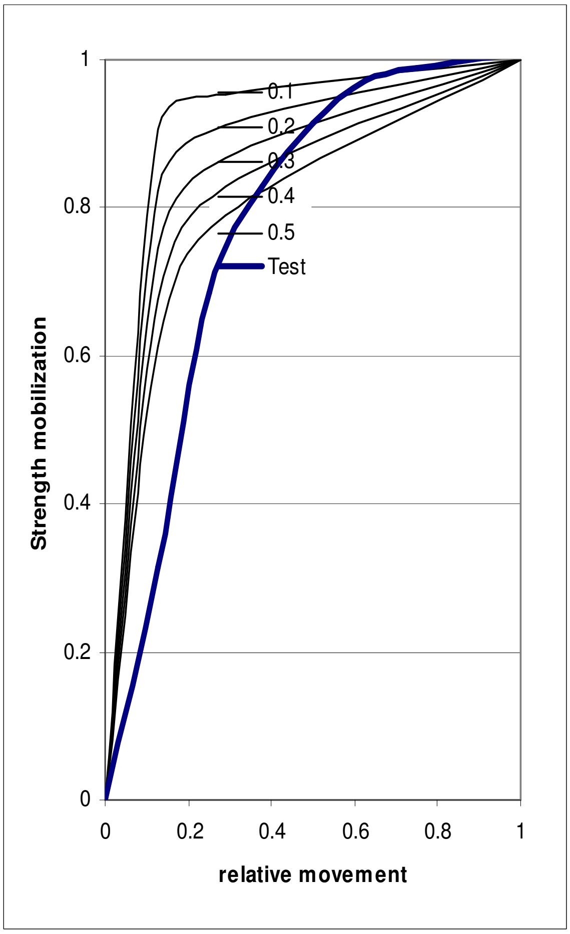

Fig. 4 Side Friction mobilization versus movement (mean value)

6. CONCLUSIONS

The results of the present data calibration by pile load tests highligth some major points:

a) The available methodologies for bearing and settlement analysis reveal some dependency on local conditions which leads to specific parameter for design. The present work showed the fundamental role of pile load testing for calibration purposes, in order to achieve higher confidence and more efficient solutions.

b) The maximum applied load to the toe during the test for the ultimate load accepted in the design, developed only a settlement of 2% of pile diameter, and so design values can only be checked by approach, since they are estimated for 5% of pile diameter. However, the results match satisfactory, if Reese & O'Neill´s (1999) diagram is used.
c) The toe bearing exponential factor (g) for settlement analysis (Fellenius, 1999) adjusted well with the load test results, both in shape and magnitude.
d) In the present case, LCPC (Bustamante & Frank, 1999) methodology is more realistic for side friction evaluations, although sligthly unconservative. Reese & O'Neill's (1999) seem to be quite far from the real situation, which may be related to the fine content percentage that affects the efficiency of friction angle evaluation.
e) Side friction load-settlement curves showed values for full mobilization that agree both with the independent range (10 to 15mm) as well the one based on type of soil and pile diameter.
f) Side friction exponential factors (Fellenius, 1999) show trend lines with similar shape, although they can reveal some dispersion.

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