Dynamic measurements of pile performance in soft mudstone

Les mesures dynamiques du comportement des pieux dans la pélite meuble

C.NORÉN, NCC (Nordic Construction Company), Gothenburg, Sweden
C.-J.GRÅVARE, Europile AB, Gothenburg, Sweden
C.AHLÉN, NCC (Nordic Construction Company), Gothenburg, Sweden

SYNOPSIS: Predictions of bored pile bearing capacity in soft rock are usally done with use of static formulas applying measured rock properties as the unconfined compression strength. The accuracy is normally low depending on large variation in measured properties and the limited validity of the theoretical formulas giving rise to unnecessary investments in piling. This report describes a method of installing piles and measurements of pile bearing capacity with dynamic methods in mudstone. Comparisons are made between measured values and static predictions using statistical methods.

1 INTRODUCTION

The Swedish contractor, NCC (former ABV) has been contracted by the Colombian Government to build a harbour on the Pacific coast in Colombia.

The project is a turnkey project and includes approach fairway, quays, shipyard, workshops, buildings for accommondation, commerce, administration etc.

The 400 m long quay is founded on prefabricated concrete piles installed in the soft mudstone forming the main gelogical formation in the area. Dynamic measurments of pile performance as well as mathematical modelling according to CAPWAP-equation were done. Comparisons are made with predicitons of pile bearing capacity with static formulas.

2 SOIL CONDITIONS

The predominating geological formation at the actual part of the Colombian west coast is a tertiary sedimentary soft rock with a depth of up to 5.000 m (the so called NAYA formation) varying from mudstone to sandstone. No crystalline rock is available in the vicinity of the port area. Due to weathering the upper layer of the tertiary formation has been transformed to a residual soil of high to medium plastic soil of clay and silt offering very difficult conditions for construction activities during the prevailing climatic conditions (precipitation up to 10 m per year).

At the chosen location of the quay the soil consists of mudstone or claystone sometimes with lenses of cemented sand or silt. Core drilling were performed along the planned quay. The core samples were classified with respect to hardness and recuperation. On selected samples the unconfined compression strength as well as the elastic modulus were determined.

The unconfined compression strength has been correlated to hardness and recuperation according to table 1.

qu MPa	Hardness	Recuperation
< 4.2	0	30 %
4.2	1	30-70 %
5.5	2	40-80 %
5.6	3	50-90 %
11.7	4	60-100 %
14.3	5	80-100 %

Table 1. Relation between hardness, recuperation and unconfined compression strength.

In fig 1 the relation between unconfined compression strength and elastic modulus is shown. As can be seen there is a considerable scatter in the results.

The natural water content is normally in the range of 20-30 % and the unit weight about 19-21 kN/m^3.

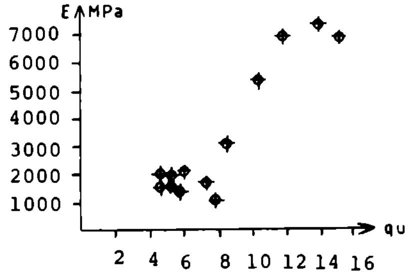

Figure 1. Relation between unconfined compression strength and elastic modulus.

3 QUAY DESIGN

3.1 General

The quay as shown in the figure 2 should give access to different types of vessels allowing a depth clearance of 9 metres at mean low water springs along the 400 m quay front. At the inside of the 80 m pier accessibility for smaller boats is planned with a depth clearance of 6 m.

For the handling activities on the quay the platform should be designed for a distributed load of 2 ton/m2 and the concentrated load of a 100 ton mobile crane.

Due to the location of the port in a severe seismic zone great attention was paid in defining the seismic criterias for the design of the quay. Due to lack of reliable data it was decided to perform an extensive seismic risk analysis. The evaluation recommended that the quay should be designed for a 200 year ground acceleration of 0.3 g in ultimate state.

Another important factor when considering different quay designs were the great tidal fluctuation (MHWS - MLWS = 3.7 m).

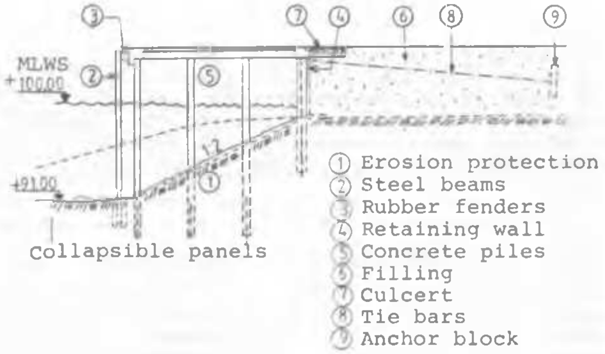

Figure 2. Section of quay

When evaluating different quay designs special weight were put on the following factors:

- Flexible performance due to seismic forces.
- Minimizing of maintenance.
- Selection of construction methods which could guarnatee high quality and minimize risky moments.

The chosen layout according to Fig 2 with vertical prefabricated driven piles were considered to fulfil the defined requirements.

The piles should resist a vertical service load of about 750 kN as well as the horizontal forces due to the seismic forces.

The piles with dimensions ø 550, ø 650 and 800 x 800 mm are prefabricated in a field factory on the site guaranteeing a good concrete quality. To resist chemical attack in the sea enviroment cement type V according to ASTM was specified. It was decided in an early stage of the projekt that dynamic measurements of pile performance should be realized to assure that the specified pile refusal was achieved.

3.2 Pile design

Maximum level of pile top was specified according to results of the borings.

The level was chosen from estimations of the permissible axial load from static formulas according to Poulos and Davis (1980), using relevant soil data. The necessary embedment length with respect to horizontal forces during earthquakes was also checked applying the expression $k_h = 150 * C_u/d$ for the bed modulus with the undrained shear strength derived from the results of unconfined compression strength tests.

Even if the soil parameters were chosen in a conservative manner it was decided to make pile loading tests by dynamic measurements on each pile. One reason was to detect the presence of weak zones not revealed by the geotechnical investigation. Another was the uncertainties in the static prediction of the permissible load for socketed piles. In table 2 are summarized the results of the different approaches made. The results are shown for one pile (pile type 1) with ø = 650 mm installed in the normal type of mudstone and for one pile (pile type 2) with ø = 550 mm installed in a more firm rock found locally in the norther part of the quay. Comparisons are made with dynamic measurments on piles in these areas. In the calculations the shaft resistance has been omitted.

Table 2.

	1 MPa	2 MPa	3 kN	4 kN	5 kN	6 kN	7 cm	8 MPa
PILE 1	6.15	1700	611	3050	2100	920	0.85	360
PILE 2	12.8	6700	910	4560	3314	1370	0.31	1800

1. Average value of unconfined compressions strength q_u.
2. Average value of elastic modulus determined on core samples Ecore.
3. Allowable base load (Q_{all}) according to Poulos and Davis

$$Q_{all} = 0.3 * q_u * A_b$$

4. Allowable base load (Q_{all}) considering the soft rock as a cohesive soil

$$Q_{all} = 9 * C_u * A_b/F_c$$

where $F_c = 3$ and $C_u = q_u/2$

5. Mobilized base load according to CAPWAP analysis (Q_{mob}).
6. Allowable base load according to statistical evaluation.
7. Pile settlement at mobilized load.
8. Calculated field elastic modulus from measured settlements.

As can be seen in table 2 there is a considerable scatter in the results.

In the preliminary specifications of the piling it was stated that the bottom of the bored hole for the pile installation should be filled with crusched stone with a thickness of at least 0.5 m. This in order to reach a suiffecent bearing capacity according to the preliminary static estimations. The dynamic measurments revealed that this procedure was'nt necessary.

4 PILE INSTALLATION

4.1 Installation precedures:

1. A launching beam was fixed to the right positon to the allready installed piles.
2. Stearing piles was driven in exact positions one to two meters into the soil.

4. Large tubes were then mounted to the stearing piles and driven into soil.

5. An auger were placed into this tubes, and augered minimum 3 meters into the soil.

6. The pile was then installed into exact position in the tube.

7. Trancducers were mounted on the pile for stress-wave measurements.

8. The piles were driven using a hydraulic hammer type Banut. The drop weight weighing 5-7 tonnes.

9. The driving stopped when the bearing capacity according to Case-method was 3 times the working load. Each pile was tested. (Total number of tested piles 320.)

10. Concrete was injected around the pile, and after 1-1.5 hours the tube was extracted. The procedure then started all over again.

Figur 3

5 DYNAMIC TESTING

All piles were dynamically tested. The signals from the transducers mounted on the pile were conditioned in a field computer called Pile Driving Analyser (PDA). The PDA also analyzed the signals using the Case method. The signals were also recorded on magnatic tape for futher analysis with CAPWAP.

The Case method is developed at Case Western Reserv University in Cleveland, Ohio. This method is using the measured forces and velocities at the transducers location to get the total resistance activated during the hammer blow. This resistance is then divided into a static and dynamic resistance using a damping constant that can be obtained by using recommended values, or by static load test or by the use of CAPWAP analysis. In this case CAPWAP analysis was performed on approximately 15 percent of the numbers of piles.

CAPWAP is a computer program utilizing the measured values and simulating the soil resistance acting on the pile. After an iteration process where the measured velocity is used as input together with the mathematical model of pile and soil a force is calculated. This force is then compared with the measured force. When good correlation is obtained between the measured and calculated force a good picture of how the soil is acting on the pile is obtained. This procedure can involve quite a few iterations (typically 40 to 50). The soil reaction on the pile can then be studied both in location and in amplitude.

In this article two piles are studied which represent piles in two different areas. Pile 1 represents normal geotechnical conditions and Pile 2 represents piles in a more firm mudstone.

During the test the piles had only resistance at the toe. The model was similar for all piles consisting of:

1. A static resistance which was activated as soon as the pile started to move.

2. A static resistance that was activated after a short movement of the pile.

3. A dynamic resistance.

For Pile 1 first static resistance was activated after a movement of approximatly 2.5 mm and had an amplitude of around 350 kN. The second static resistance started to be activated after a movement of the toe of approximately 3.5 mm and had a stiffness of 1350 kPa/mm. The maximum value activated on second static resistance was 1760 kN.

The dynamic resistance consisted of a viscous damping with a value of around 500 kN/m/s giving a maximum dynamic resistance of around 800 kN.

For Pile 2 (in the firmer mudstone) the first static resistance was activated after a movement of 1 mm and had an amplitude of around 1000 kN.

The second static resistance started to be activated after a movement of approximatly 2 mm and the stiffness was 6940 kPa/mm. The maximum activated second static resistance was 2300 kN.

The viscous damping value was 1050 kN/m/s giving a maximum dynamic resistance of 1150 kN.

The two different resistances can be explained by the way the pile has been installed. During the drilling some soil has been loosened giving the first static resistance during driving. When the pile toe then moves through this looser soil into the undisturbed mudstone the second resistance is activated. The dynamic resistance is approximately 0.2-0.3 of the total static one.

Typical output of the CAPWAP-analysis can be seen in Figure 4.

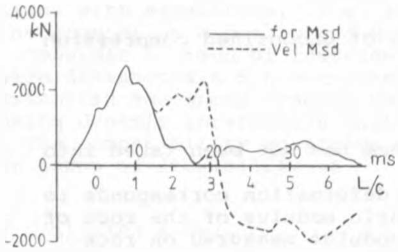

Figure 4a. Measured force and velocity Pile 1.

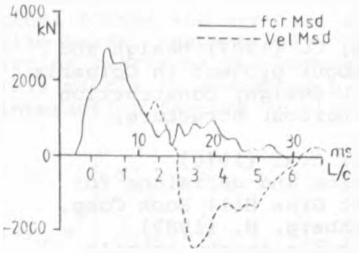

Figure 4b. Measured force anc velocity Pile 2.

6 TYPICAL BEARING CAPACITY - RELIABILITY ANALYSIS

As indicated earlier the bearing capacity of the piles can be written in the form:

$$Q_{all} = K * q_u * A,$$

where K is a constant and q_u is the average value of the measured unconfined compression strength. However there is a considerable difference in the value of K depending whether the mudstone is regarded as a soft rock or a cohesive soil. To determine a suitable value of the bearing capacity a reliability analysis was made upon the data obtained from the pile dynamic measurements and rock investigations.

With the different variables modelled as stockastic ones and with a reliability index $\beta = 4.8$ (referring to a probability of reaching the achieved value in the range of 10^{-6}) the constant was determined to be around 0.45.

7 CONCLUSIONS

1. The possibility to make accurate predictions of bearing capacity of socketed piles in soft rock based on measured rock properties are limited.
2. Dynamic measurements analysed with CASE and CAPWAP methods will give a good picture of a mobilized pile bearing capacity.
3. The applied installation-techniques using prefabricated piles placed in augered rock and driven through the softened rock will imply reduced settlement in comparison with cast in situ piles at the same stress level.
4. The allowable pile load (at a probability of reaching the achieved values of 10^{-6}) with the applied installations-tecnique and this type of rock can be axpressed:

$$Q_{all} = K * q_u * A_b \quad K = 0.45$$

where

q_u = Average value of unconfined compression strength

A_b = base area

The shaft resistance has not been taken into account.

5. The measured deformation corresponds to an equivatent elastic modulus of the rock of about 1/5 of the modulus measured on rock cores.

REFERENCES

Apleberger, L., Norén, C. (1987) Design and construction of harbour project in Colombia. Baltic Conference on Design, Construction and Maintenance of Harbour Structure, Norrköping.

Benjamin, J.R. Cornell C.A. (1970) Probabilitis, statistics and decisions for civil engineers. Mac Graw Hill Book Comp.

Broms, B.B. and Bredenberg, H. (1982) Applications of stresswave theory on pile driving. A state-of-the-art report. Procedings of the Seventh Southeast Asian Geotechnical Conference, Hong Kong.

Cole K.W., Storud M.A. (1977) Rock socket piles at Coventry Point, Market Way, Coventry Piles in weak rock, The Inst of Civ Engineers, London.

Goble, G.G. Liking, G.E. and Rausche, F. (1975) Bearing capacity of piles from dynamic measurements. Final report, Dept. of Civ. Engineering, Case Western Reserve University, Cleveland, Ohio.

Hermansson, I. and Grävare, C-J. (1978) Static bearing capacity of piles from dynamic measurements. Väg- och Vattenbyggaren No 8-9.

Olsson, L. (1986) Användning av -metoden i geotekniken. Royal Institute of Technology, Stockholm. (Doctorial Thesis).

Poulos H.6, Davis E.H. (1980) Pile Foundation analysis and design. The University of Sidney, John Wiley and Sons.

Rausche, F. (1970) Soil response fron dynamic analysis and measurements on piles. Ph.D. Thesis Case Western Reserve University, Cleveland, Ohio.

Thost-Christensen, P., Baher, M.J. (1982) Structural reliability theory and its application. Stringer-Verlag.

Tomlinson M.J. (1977) Discussion: Session 2-rocks other than chalk. Piles in weak rock, The Inst of Civ Engineers, London.

Wilson L.C. (1976) "Tests of bored and driven piles in cretaceous mudstone at Port Elizabeth, South Africa. Piles in weak rock. The Inst of Civ Engineers, London 1977.