Evaluation of the coefficient of earth pressure at rest K_0 of a soft clay in Rio de Janeiro by using earth pressure cells, dilatometer tests and laboratory tests

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ABSTRACT: The coefficient of earth pressure at rest K_0 used in geotechnical engineering analysis depends on the characteristics of its application. There are many theories for predicting lateral earth pressure; some are empirically based, some are analytically derived. In this paper a K_0 evaluation for the Rio de Janeiro soft clay based on field measurements with earth pressure cells, in situ testing due to dilatometer and laboratory tests is presented. Measurements and tests were carried out within an execution of a geotechnical load test on a construction site of the ThyssenKrupp CSA, in the Sepetiba Bay. The results from different methods suggest K_0 in a range of 0.8 to 0.9

KEY WORDS: K_0, soft clay, DMT, laboratory tests.

1 INTRODUCTION

ThyssenKrupp CSA is building a new steel mill in the coastal location of Sepetiba in the state of Rio de Janeiro. The mill will have an annual capacity of 4.4 million tons of steel formed in slabs with a single weight up to 35t. The steel slabs will be stored intermediately at the Slab Yard, an area of 50,000 m². Overhead bridge cranes will be erected for the deposit and the retrieval of the steel slabs. The cranes have to be founded by piles on the 35-meter-deep seated rock. The rock is overlayed with sedimentary soils in an alternating stratification of sand and clay layers. The upper 17 m of the soil is mainly composed of a clay layer with soft to very soft consistency.

Caused by the required capacity of the storage areas high area loads have to be considered. Various solutions for the foundation of the storage area were studied. At this point it was important to clarify that any soil mass displacement due to settlements or undrained behavior of the soft soil had to be considered as a relevant loading hypothesis for the piles of the over head bridge cranes.

Solutions were classified based on the soil mass displacement due to storage load. Rigid and semi-rigid solutions were ruled out for economic and time reasons. To found the storage areas without driving piles or improving the soil a flexible solution with the following reflections was investigated:

- The production of steel slabs during the start up of the mill will not require the full storage capacity. Due to a coordination of the distribution of steel slabs, areas can be loaded in steps.
- As consolidation proceeds, the shear strength of the soil increases within the steps. Further load steps will cause soil mass displacements to a lesser extent as in case the total load would be applied at once. The resulting lateral thrust and flow pressure of the soil will be

smaller. Thus, lateral loading and movement of the piles of the overhead bridge cranes are expected to be less extreme.

- A simple flexible reinforced concrete slab will equalize the settlement of the storage areas and avoid local failures of the supporting soil. Therefore the stability of the stacks with partially 900°C hot steel slabs is ensured.
- The settlement depression on the surface can be equalized by backfilling.

The low rate of consolidation settlement in the saturated clays of low permeability will be accelerated by vertical drains. Hence a maximum of the consolidation settlement can take place within the provided time.

This solution was the most economical one and seemed to be simple. However a conventional geotechnical calculation based on prescriptive measures and comparable experience could not ensure its suitability. Values of deformations and forces of ground and structural elements had to be established essentially due to a large scale test. The test and its preliminary results were published by Mühl *et all* (2009). Figure 1 shows a bird view of the test field. Figure 2 shows one of the monitoring cross sections of the system and the instrumentation.

Figure 1. Bird view of the test field.

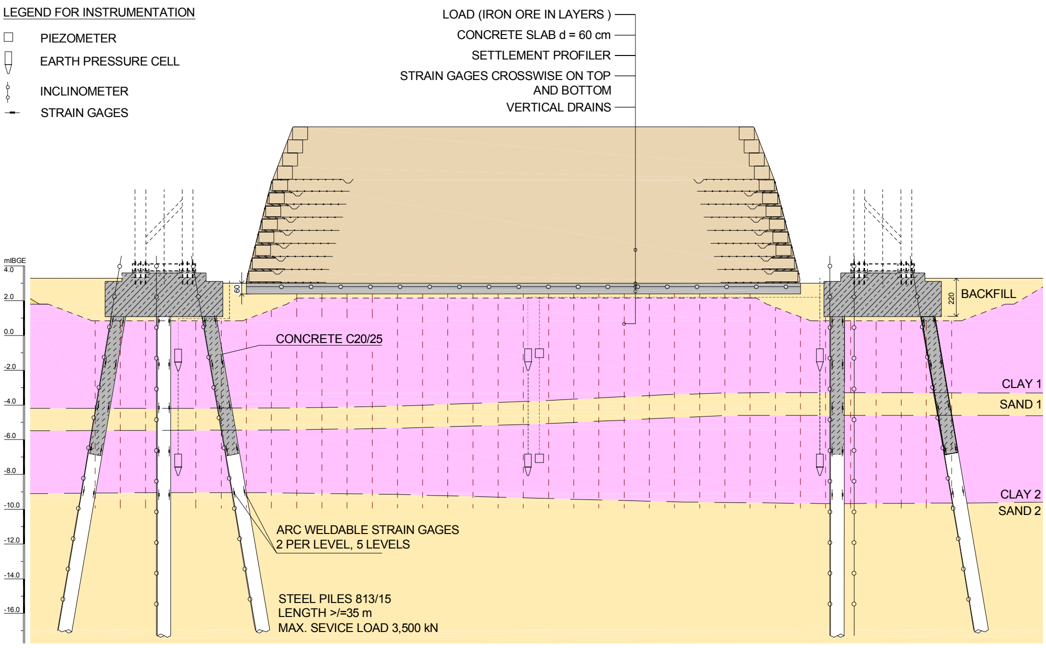

Figure 2. Cross section system and instrumentation of large scale test.

2 SLAB AREA TEST FIELD

2.1 Soil Investigation

In order to provide geotechnical information on site area, SPT, vane shear strength, CPT-u and

DMT tests were carried out. In addition, Shelby samples were collected to perform oedometer and UU triaxial tests.

2.1.1 Soil description

The original ground surface was around 1.4 to 1.5 mIBGE. Meanwhile, the area was land-filled to a level of ~ 3 mIBGE. The symbol mIBGE indicates the ground level, in meters, related to the IBGE reference.

Beneath the made ground follows a very soft, sometimes soft to stiff soil layer down to a depth of about 4 to 7 m. The layer consists of silty clay, sometimes with a slightly sandy part with a fine grain (d < 0.06 mm) of more than 85 %.

Based on various lab tests in different parts of the site the natural water content is between w = 105 to 138 %, the liquid limits are between w_L = 64% and 88% and the plasticity index I_P = 26 to 40 %. With the organic content (loss on ignition) between 4.8 to 7.3 % the clays are classified as little humus to humus. The soil can be described as organic clay.

CP-tests were carried out to investigate the clay layers. The tip resistance varies between 0.15 and 0.40 MN/m². The undrained shear strength lies in the range between 5 and 30 MN/m².

Beneath the upper clay layer (CLAY 1) sand was encountered in most soundings in a thickness of about 1 to 3 m. The sand mainly shows blows of N_{30} = 8 to 20 and a tip resistance of mostly about 5 to 8 MN/m². This corresponds to a medium density.

Below the sand layer follows a second clay layer (CLAY 2). It reaches depths of about 12 to 18 m. The layer is largely similar to the upper clay layer. Regarding the results of the SPT's with blows up to N_{30} = 3 to 8, sometimes N_{30} > 8, and a tip resistance of 0.4 to 0.8 MN/m², a soft to sometimes stiff consistence can be observed. The undrained shear strength is between 20 to 30 kN/m².

Below the second clay layer to the depth of approximately 18 to 28 m is a changing sequence of medium to coarse sand frequently with silty parts and clay in varying thickness. The sand shows mostly a loose to medium density. The clay parts are mostly soft to stiff. In some areas very soft consistencies are possible.

Below the "mixed layer" is either a gravely silty coarse sand or a sandy silt. The silt shows a firm to hard consistency. The sand with N_{30} > 15 and a tip resistance of more than qs = 10 MN/m² has a medium density. This layer is rich in mica, an indication for weathered bed-rock. The hard rock follows in a short distance or directly at the bottom of this layer.

Figure 3 illustrates one SPT, taken in the middle of the test field.

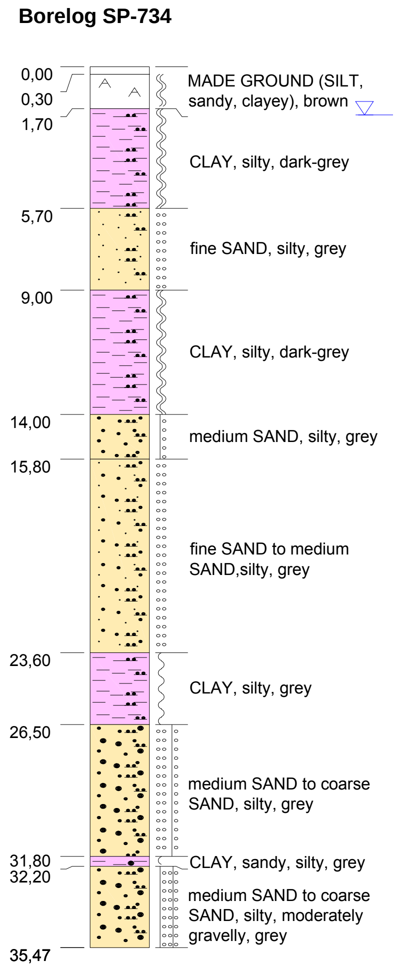

Figure 3. Soil profile on slab area.

2.1.2 DMT Tests

There were carried out sixty-one DMT tests. The readings where taken in 11 boreholes each meter to a maximum depth of 12 m.

The earth pressure coefficient K_0 was estimated using the equations proposed by Marchetti (1) and Lunne *et all* (2).

$$K_0 = \left(\frac{K_D}{1{,}5}\right)^{0.47} - 0{,}6 \qquad (1)$$

$$K_0 = 0.34 K_D^{\;0.55} \qquad (2)$$

The results are shown in Figure 4 and summarized in table 1.

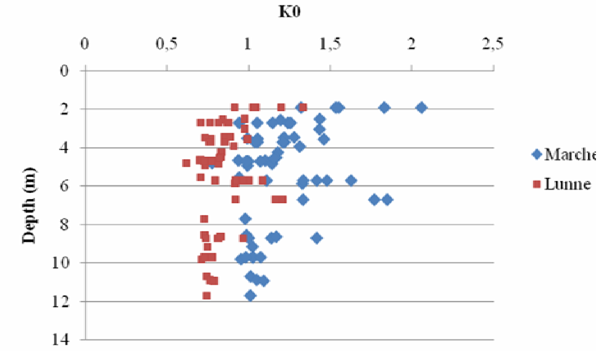

Figure 4. K_0 evaluated by DMT tests.

Table 1. K_0 evaluation by DMT tests

Method	Average Values	
	Layer 1	Layer 2
Marchetti	1.26	1.06
Lunne	0.88	0.77

2.1.3 Laboratory Tests

On the area of the test field oedometer tests on samples collected in Shelby tubes, in both first and second clay layer, were carried out. No triaxial tests were performed on these samples.

In neighboring areas of the construction site, with comparable subsurface conditions, oedometer and CIU triaxial tests were executed on the same clay layers. The friction angles obtained on CIU triaxial tests performed on the neighboring were considered the same for the soil layers of both areas.

With the test results, the earth pressure coefficient at rest was estimated using the equation proposed by Mayne and Kulhawy (3). Results are shown in Table 2 and 3.

$$(K_0 = (1-\sin\phi')OCR^{\sin\phi'}) \qquad (3)$$

Table 2. K_0 evaluation by laboratory tests in test field area

Sample	Depth (m)	OCR	ϕ'	K_0
1	4.3	3.5	23	0.99
	12.9	1.6	21	0.75
2	3.3	2.7	23	0.89
	9.7	4.9	21	1.13
3	3.3	3.8	23	1.03
	10	1.0	21	0.64
4	4.3	1.2	23	0.65
	12.3	2.4	21	0.88
	Total average			0.87
	Average layer 1			0.89
	Average layer 2			0.85

Table 3. K_0 evaluation by laboratory tests in neighboring areas

Sample	Depth (m)	OCR	ϕ'	K_0
1	3.25	1.3	23	0.68
	11.25	1.6	21	0.76
2	3.75	2.0	23	0.80
	12.25	2.2	21	0.85
3	2.25	1.1	23	0.63
	9.75	5.8	21	1.20
4	3.25	1.0	23	0.61
	9.25	2.2	21	0.85
5	3.25	1.1	23	0.63
	9.25	7.6	21	1.33
6	2.75	1.0	23	0.61
	10.25	10.4	21	1.49
	Total average			0.87
	Average layer 1			0.66
	Average layer 2			1.08

Figure 5 shows estimated values of K_0 due to the equation by Mayne and Kulhawy in relation to the depth the samples where taken from.

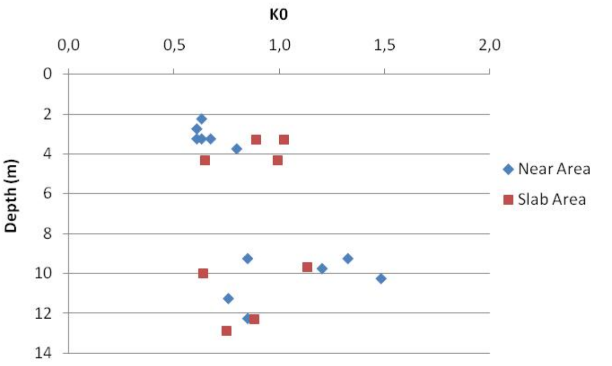

Figure 5. K_0 evaluated by laboratory tests.

2.2 Earth pressure cells

In the upper and lower clay layers, 12 vibrating wire pressure cells with integrated piezometers and five vibrating wire piezometers were installed. The instruments were located beneath the middle of the slab and beneath the borders of the slab in front of the piles (refer figure 9). The concrete slab was loaded with iron ore in 5 equal load steps up to a total area load of 250 kPa. Each load step was hold between 40 to 60 days.

Figures 6, 7 and 8 show the total pressure and pore water pressure measured by earth pressure cells and piezometers. Figure 9 shows a ground floor plan of the test field with the location of the earth pressure cells and piezometers.

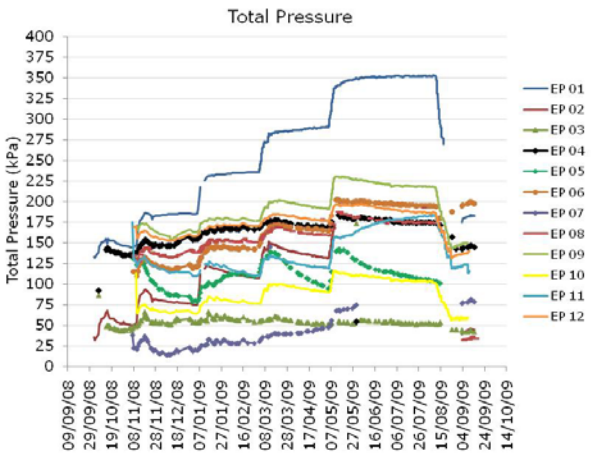

Figure 6. Total pressure measured by pressure cells.

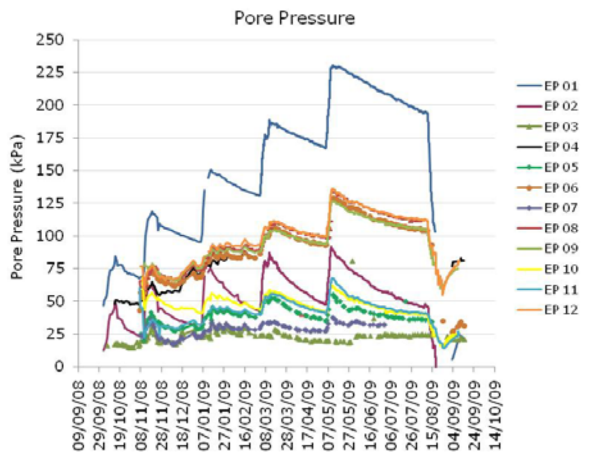

Figure 7. Pore pressure measured by pressure cells.

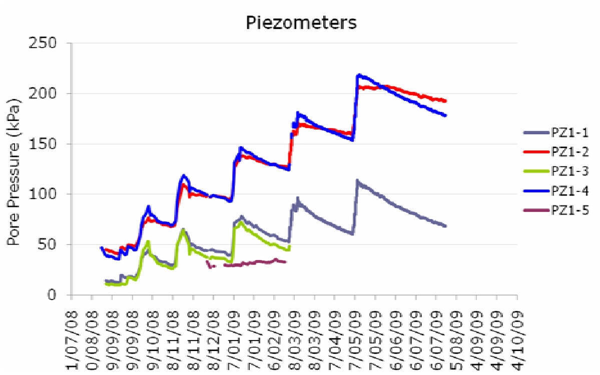

Figure 8. Pore pressure measured by piezometers

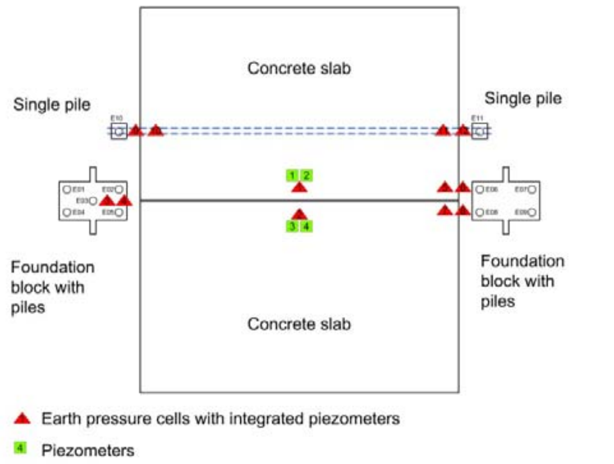

Figure 9. Ground floor plan of test field with location of pressure cells and piezometers.

The coefficient of earth pressure K was evaluated using pressure cells and piezometers data by the ratio of the effective horizontal stress increment and the total vertical stress increment. To each load increment, K was evaluated for each cell. The results are shown in Table 4.

Only at the earth pressure cells beneath the middle of the slab can be assumed that no lateral movement of the soil takes place. Therefore only for the location of EP1 and EP 2 the estimated coefficient of earth pressure can be assumed as the at-rest-coefficient K_0.

The earth pressure cells beneath the borders of the slab in front of the piles are subjected to lateral movement of the soil. The value of the movement depends on the constraint due to resistance of the piles and the soil. The coefficient can not be evaluated as K_0.

Table 4. K evaluated by pressure cells.

Pressure Cell	Location beneath slab	Depth (m)	K
EP1	Center	10.30	0.81
EP2	Center	3.8	0.97
EP3	Left border	2.4	0.95
EP4	Left border	8.1	1.20
EP5	Right border	5.1	1.89
EP6	Right border	10.2	1.20
EP7	Right border	5.1	1.70
EP8	Right border	10.2	0.87
EP9	Left border	10.15	1.15
EP10	Left border	10.15	1.38
EP11	Right border	5.1	1.25
EP12	Right border	10.2	0.66
Average on border, layer 1			1.45
Average on border, layer 2			1.08
Center, layer 1			0.97
Center, layer 2			0.81

3 DISCUSSION

Results for K_0 obtained by the three methods show dispersion. Figure 10 shows the resulting K_0 values distributed over the depth. It has to be noted that the coefficient of earth pressure derived from pressure cells is not K_0 on the strict sense.

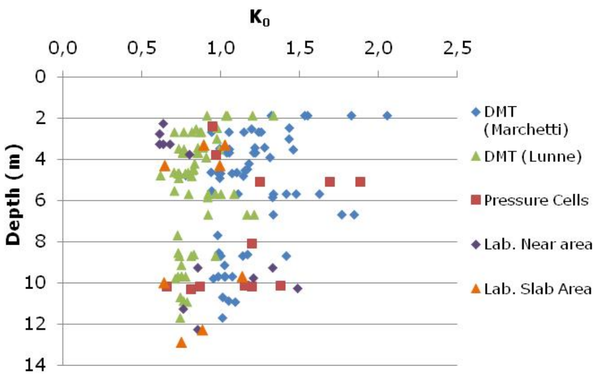

Figure 10. K_0 evaluation by all employed methods

De Campos (2006) studied the soft clay of Santa Cruz and obtained a K_0 of approximately 0,62 performing triaxial tests. This value was obtained with controlled stress paths and radial strain measures. Samples tested were collected from 3.5 to 4.0 m depths. The OCR to these depths was 1.95 and the effective friction angle was 26.3°. Applying equation (3) to the data a K_0 of 0.75 is obtained.

Values figured out by DMT using equation (1) were considered higher than expected and not suitable. In addition, the results show a great dispersion. Equation (2) showed results more consistent and considered suitable.

Despite the lack of shear strength laboratory tests on the test field area and using values of a comparable neighboring areas equation (3) leads to results consistent or at least reasonable. From the results from the samples taken at the neighboring areas we expected higher values of OCR and consequently of K_0 in shallow levels. Results of measurement with earth pressure cells in front of the piles are influenced by soil movement and principal stress directions. The coefficient will not describe the at-rest earth pressure. But the results based on the data of the two pressure cells beneath the middle of slab can be considered as approximations of K_0.

4 CONCLUSION

Three methods to evaluate K_0 of a soft clay in Rio de Janeiro were employed.

Based on the results, it is suggested that K_0 for clay layer 1 is around 0.9 and for clay layer 2 is around 0.8.

The use of equation (1) by Marchetti was considered unsatisfactory. The equation (2) by Lunne was considered satisfactory.

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