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Earth pressure on braced flexible walls – Model tests and field investigations

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SYNOPSIS : This paper concerns the behavior of braced flexible walls. Field observations on two different sites of the subway system in Lyon are discussed. The Rido software, which utilizes the reaction modulus of the soil is used to predict the soil behavior. The measured displacements, differential earth pressures, bending moments and load in struts are compared to the calculated ones.

1 INTRODUCTION

Many kilometers of the subway tunnels in Lyon (France) have been constructed by the cut and cover method using temporary sheet piles and cast in place or precast concrete walls.These walls are supported by 2 or 3 levels of passive struts.The construction of this subway system started 20 years ago and it is still in progress. From the very begining of this construction, an extensive instrumentation and monitoring program is being carried out.

In order to study the behavior of these temporary retaining structures and to improve their design, a particular attention has been given to the prediction of the movements, bending moments, earth pressures and load in struts, by using the Rido computer program (Fages & al 1971) based on Winkler's hypothesis.

This paper outlines the discordance between predicted and measured values by analysing the results from two sites with different geological conditions. We also discuss the influence of the seepage flow at the bottom of the sheet pile on the diminuation of the passive earth pressure.

A series of experiments on a bidimensional small scale model confirm the limitations of the reaction modulus method.

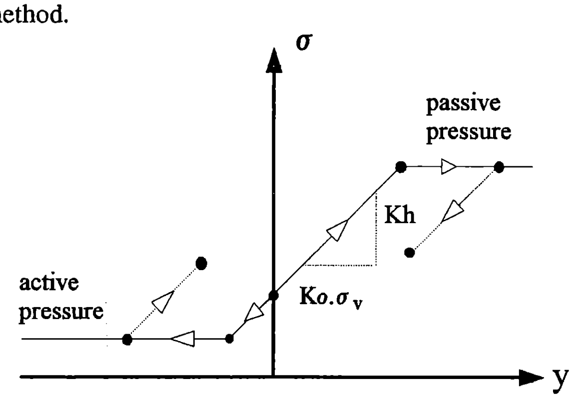

Figure 1. Load-deflection relationship of the software Rido.

2 CALCULATION USING REACTION MODULUS

The Rido program was especially developped for the construction of the first subway line and is now of general use in France. In Rido, the earth pressure is considered separately on every side of the retaining wall. The reaction modulus of the soil is limited by active and passive pressures and a hysteretical load-deflection relationship (figure 1).

3 EXPERIMENTAL SITE (COURS GAMBETTA)

3.1 Site and construction description

The excavation is situated on the east side of Rhône river. Under 4 m of recenty deposed soil we find 18 m of alluvial sandy gravel and a sandstone substratum. Figure 2 presents the excavation profile : the tunnel on the alluvial penetrates about 5 m into the water table. The high permeability of alluvial (k about 10^{-2} m/s) does not allow the lowering of the water table by pumping, therefore a water tight wall (0.6 m thick) fill in with a bentonite-cement slurry is constructed. The embedment depth of this wall in the sandstone substratum is 3 m. The sheets piles

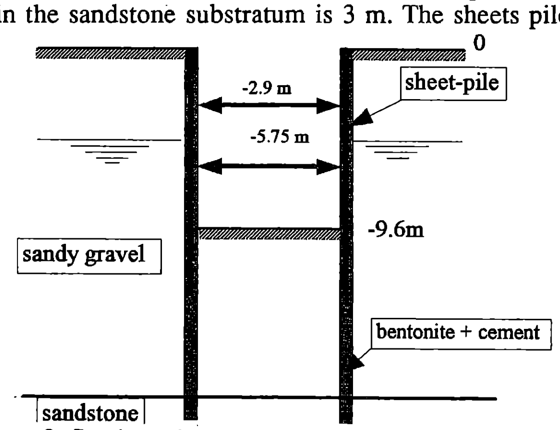

Figure 2. Section of the excavation (cours Gambetta).

are 12.70 m high and completely embedded in the slurry wall. After the excavation the sheet piles are supported by two levels of struts designed to limit the lateral movements toward the excavation.

As the sheet piles are monitored, the second level of struts are placed with 2.5 cm of slack to permit a sufficient bending of the sheet piles.

The characteristics of alluvials are measured by direct shear field tests and back analysis on similar materials (Kastner & al 1984) :

bulk density		$\gamma/\gamma_w = 2.05$	
submerged density		$\gamma'/\gamma_w = 1.25$	
friction angle	$\phi = 33°$	cohesion	$c = 24$ kN/m^2
reaction modulus		$k_h = 10^5$ kN/m^2	

3.2 Comparison of calculated and measured values

Inclinometers are installed on the sheet piles and displacements at the top of the piles are measured by surveying instruments. Extensometric equipment permits us to determine the distribution of bending moments of the sheet pile and also the load in the struts.

Figure 3 presents the measured displacements on the 2 last steps of construction : we observe a considerable increase of displacement when the excavation depth increases from 6.5 to 9.5 m.

A back analysis permits to adjust the set of soil parameters used in Rido program in order to fit the calculated and measured displacements at each construction stage.

We observe that the values calculated by the software Rido are highly influenced by the cohesion term while the variation of reaction modulus does not affect them.

If this adjustment seems to be acceptable for the deflection curve, but it is almost impossible to have simoultaneously correct values for the load in the struts (figure 4). The calculated load is about 50% less than the measured one.

In order to explain this discrepancy we study the calculated and the measured earth pressure values. The measured earth pressure curves are obtained by the fourth derivative of deflection curves. These curves imply a high concentration of stress just at strut levels. Such a high stress concentration can be justified by a differential displacement, between the top and the bottom of the sheet pile, which occurs at the second step of excavation. This concentration can not be calculated by Winkler's hypothesis which considers only the local displacements and not the differential ones. We note that the difference between calculated and measured load in struts equals the area between two earth pressure curves.

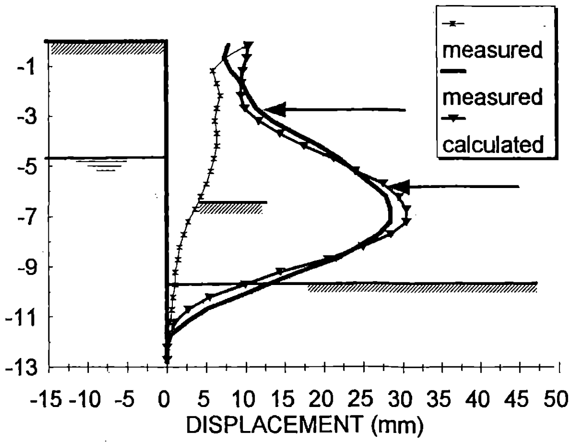

Figure 3. Comparison between calculated and measured displacements.

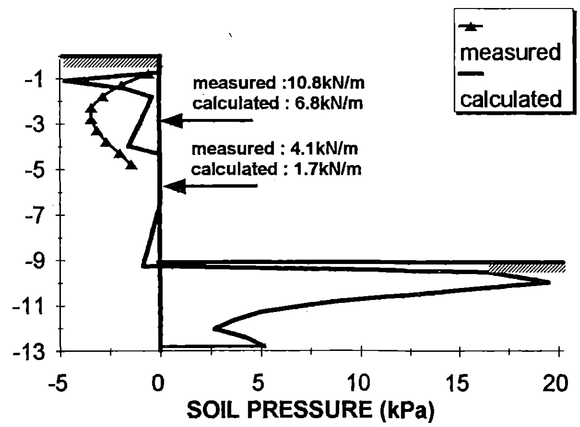

Figure 4. Comparison between calculated and measured differential earth pressures.

4 EXPERIMENTAL SITE (GORGE DE LOUP)

4.1 Site and construction description

The excavation, 9 m wide and 8 m deep, is supported by two cast in place retaining walls made of reinforced concrete 10.5 m high and 0.6 m thick (figure 5). these walls are restrained against lateral movements by two series of passive struts made of steel H-shaped sections. Before the work, the water table varied between 4 and 5 m below the ground surface.

A preliminary hydrogeological study showed that the groundwater level might rise when intersected by these cast in situ walls. As a result, the retaining walls were designed with a reduced depth of embedment, and this in turn led to a heaving risk of the excavation floor.

The displacement of the retaining walls is monitored by inclinometers and surveying instrumentation. The load in struts is measured by extensometers. Furtheremore the water level during the excavation is surveyed by four piezometers. Four pore pressure measuring cells are placed at the foot of the pile to evaluate the uplift gradient.

On this site, the soil consisted of a succession of sandy silt layers with fairly similar identifying characteristics. The liquid limit of these silts varies between 25 and 30% with a plasticity index close to 5%. It should be noted that the natural water content of these silts is always close to their liquid limit, which explains why they are so sensitive to disturbance, especially during earthwork. Mechanical

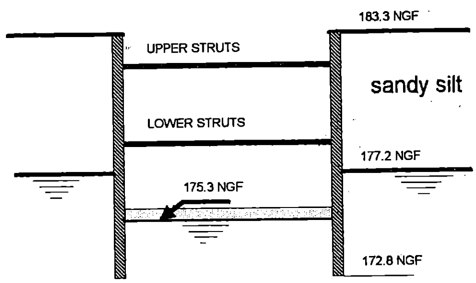

Figure 5. Section of the excavation (Gorge de Loup).

properties are homogeneous within a depth of 9 m The characteristics, deduced mainly from triaxial tests, vane tests and pressuremeter tests, are as follows :

- between 0 and 9 m :
  - undrained shear strength cu = 70 to 90 kPa
  - shear strength parameters with respect to effective stress $\phi' = 27°$ and $c' = 0$
  - Menard limit pressure Pl = 0.2 to 0.4 MPa
  - Menard elastic modulus Ep = 1 to 2 MPa
- beyond a depth of 9 m, the soil has slightly better characteristics :
  - cu = 100 to 150 kPa
  - $\phi' = 37$ to 40° and $c' = 0$
  - Pl = 0.7 to 0.9 MPa Ep = 3.5 to 4 MPa

4.2 Comparison of calculated and measured values

In figure 6 we present the comparison between the calculated deflection curve and the measured one during the two last steps of excavation. These curves have the same shape with a 3 mm translation. This difference could be attributed to the asymmetric load on the excavation (movement and parking of earthwork machinery on the western side very close to the excavation) which causes a general movement of the two parallel walls. This movement can not be calculated by Rido (Kastner 1992).

The first series of calculations are based on a reaction modulus value given by pressuremetric modulus. This method leads to very high displacements in comparison with those measured, especially for the first steps of the excavation. A back analysis was carried out and the modifications to the reaction modulus value that produced a calculated profile similar to the recorded profiles are described.The final modulus value should be about 10 times greater than the value obtained by pressuremetric modulus (from 2 to 4 10^4 kN/m^3). This proves that the reaction modulus value which is not an intrinsic parameter of the soil, is very difficult to estimate.

4.3 Influence of seepage flow at the toe of the excavation

When the excavation depth reaches its maximum, the pore water pressure at the bottom of the retaining walls shows that the hydraulic head decreases by 1.4 m for 2.5 m of embedded length (mean hydraulic uplift gradient = 0.56).

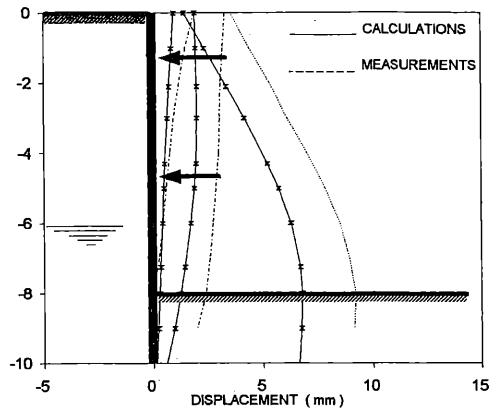

Figure 6. Comparison between calculated and measured displacements.

The safety factor for the heave of the bottom of the excavation is : $f = i_c/i = 1.7$ (i_c = critical gradient and i = actual gradient).

Seepage flow does not create any damage at the bottom of the excavation in this site, however we analysed its effect on the decrease of the passive earth pressure and on the global stability of the retaining walls. In this manner, a re design leads to decrease the passive coefficient (kp) about 40%, in order to obtain a good concordance between calculated and measured deflection curves.

In fact, the decrease of the passive earth pressure can not be detemined only by the mean gradient, and it is necessary to take into account the real flow lines shape of the seepage around the embedded length on the passive prisme.(Soubra & al 1992)

4.4 Load in the struts

For the second site (Gorge de Loup) it is also possible to obtain a correct adjustment between measured and in situ displacements by back analysis. However the calculated loads in the passive struts are basically different from those measured. The software (Rido) underestimates the load in the upper struts. The difference between these values is about 40% at the moment the struts are placed and the difference reaches 54% a few weeks after the maximal excavation.

On the deflection curves, at the upper level, there is a point without any displacement but the bottom of the wall moves towards the excavation : it seems clear that because of these differential displacements, we obtain a concentration of active earth pressure at the upper struts level. This phenomenon increases after the end of excavations because of the shrinkage of the soil at the excavation level, and because the displacement of the bottom of the sheet increases. Differential movements of the retaining walls which cause a transfer of load are not calculated by reaction modulus design.

5 MODEL TESTS

To confirm our field experiments, small scale laboratory tests on a bidimensional model, with 2 level of struts are

Table 1. Experiments on model.

test	prestress in the 1st strut kN/m ml	prestress in the 2nd strut kN/m ml	stiffness of struts kN/m ml
A	0.38	0.33	83000 stiffs
B	0.36	0.40	816 flexibles
C	2.07	2.70	816 flexibles

investigated. The soil is simulated by the material of Taylor-Schneebeli : rustproof steel rollers of 3,4,5 mm diameter ($\gamma_d/\gamma_w = 6.1$ $\phi = 21°$ and $c = 0$)

The model pile is made of duralumin with 0.08 m width, 0.012 m thick and 0,805 m high. Its flexibility is in the range of the semi-flexible retaining walls such as the retainings to be found in the subway system in Lyon.

The horizontal movements of the wall at its top and bottom are measured with mechanical dial gauges. The deflection of this pile is measured on 20 levels by 30 strain gauges mounted on both sides of the wall. The deflection curve permits to obtain the displacement and the soil differential pressure curves by the least square method, double integration and double differentiation.Three sets of experiments are discussed (table 1).

All tests are carried out in a similar manner. First the "soil" on the outside of the pile is dredged away to a depth of 0,15 m. After the fixation of the first strut, at 0.05 m from the top of the pile, the excavation is continued in steps of 0.1 m to a depth of 0.4 m then the second strut is fixed at 0.25 m and the excavation is continued until the collapse caused by a lack of passive pressure at 0.63 m.

5.1 Comparison of model results and calculations

The reaction modulus design is based on the following values :

$k_{a\gamma} \cos\delta = 0{,}39$ $k_{p\gamma} \cos\delta = 3{,}1$ $\delta/\phi = 1$

$K_h = R_p\ \sigma_v$ and $R_p = 1500\ m^{-1}$

The soil reaction modulus (K_h) which can not be measured is determind by back analysis on two preliminary tests and then kept constant.

When the struts are passive, there is a difference between the calculated earth pressure and the measured one : the pressure concentration at strut levels (soil arching) is completely neglected by the reaction modulus design which calculates a triangular distribution. This difference disapears when the struts are active (figure 7). The concentration of pressure on the calculated earth pressure profiles for active struts is only due to their prestress, not to the soil arching which can not be calculated by the reaction modulus method (Masrouri 1986).

Figure 8 shows the measured displacements on the 2 last steps of excavation for the experiment with passive struts. We observe an important increase of displacement at the bottom of the pile. The earth pressure increases considerably at the same time. This pressure concentration is completely neglected by Rido.

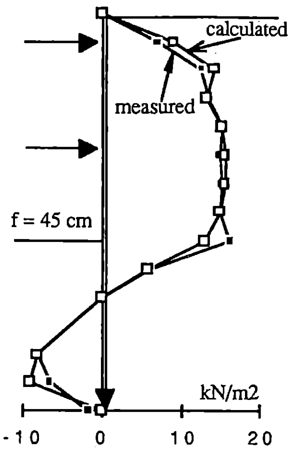

Figure 7. Comparison between calculated and measured earth-pressure.(two actives struts,test C).

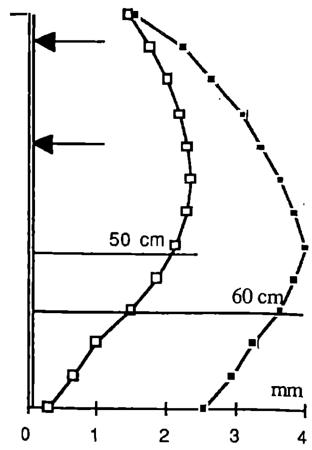

Figure 8. Increase of the displacements on the last steps of excavation.

6 CONCLUSION

Several field investigations carried out on flexible retaining walls of the subway system in Lyon, completed by laboratroy model tests, allowed an analysis of the behavior of these strutted structures.

The comparison between experimental results and back analysis by the Rido software outlined the difficulties and limits of the Winkler's hypothesis :

- the calculations highly underestimate loads in the passive struts because of the underestimation of the soil pressures on the fixed upper part of the wall. This hypothesis is not able to predict such a pressure redistribution due to a differential movement of the top and the bottom of the structure ;
- the reaction modulus value which is not an intrinsic property of the soil is difficult to estimate. In particular for silty soil, this value is very pessimistic when given by pressuremetric modulus.

Finally, the importance of the decrease of passive earth pressure due to the uplift seepage on the base of the excavation is outlined.

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