Double Anchored Tie-Back to Avoid Soil Deformation

Ancrage Double pour Eviter Déformation du Sol

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SYNOPSIS The stability of the vertical slopes of deep excavations is normally investigated admitting an elastic and / or ideal plastic behavior for all the materials involved. Such a consideration was made during excavation for the Largo da Carioca station of the Rio de Janeiro Subway System. But when the excavation reached a certain depth, very accelerated settlements were observed in the buildings near the Largo da Carioca site. It was assumed that as a result of the high foundation loads of these buildings and the stress relief caused by the deep excavation, the subsoil near the bottom of the pit plastified or suffered adverse stresses.

The solution adopted to compensate for this phenomenon consisted of prestressing the affected area, using vertical tie backs, anchored at great depths. Due to specific site conditions, double anchored tie-backs were used for the first time.

The efficiency of this construction method was confirmed by observations made during and after the excavation.

INTRODUCTION

Figure 1 shows the geometrical layout of the "Largo da Carioca" subway station excavation in Rio de Janeiro, Brazil.

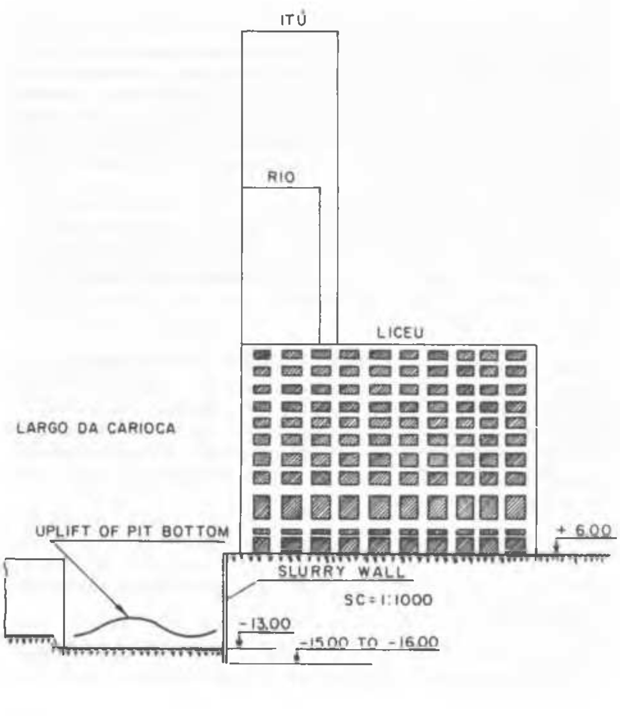

Fig.1 - Layout of Excavation Site

The excavation reached a final depth of about 20 meters, adjacent to up to 100m rised multistoried buildings. The bottom of the pit was deeper than any foundation level of those buildings.

The excavation represents a certain load case, which causes stress modifications up to yielding when the critical depth is surpassed. From elasticity theory we know that the critical depth is aproximately:

$$z_{crit} = H \{ 1 - \frac{1}{\pi} [\cot g \, \varphi' - (\pi/2 - \varphi')] \} \quad (1)$$

when H is assumed height of ground and surcharge. This fact could be confirmed by the sudden increase of the settlements when the excavation surpassed this level. These settlements could only be stopped by backfilling. Fig. 2 shows a period of very accelerated settlements when excavation partly proceeded from the -7,50m level to the -13,00m level. Therefore we believe, that design of such large scale pits should not be based on rupture criteria alone but should take into acount the plastification of the subsoil.

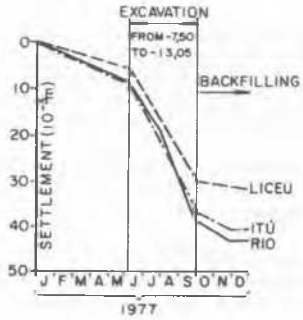

Fig.2 - Settlements Evolution

DESIGN PRINCIPLES

From elasticity theory we find the critical load

$$q_{crit} = \frac{\pi \cdot \gamma \cdot z}{\cot g\ \varphi' \cdot (\pi/2 - \varphi')} \qquad (2)$$

For example when $z = 3m; \gamma = 12\ kN/m^3; \quad \varphi' = 35^o$ we find:

$q_{crit} = 168$ kPa

and the ultimate load is:

$q_r = 493$ kPa $\simeq 3 \cdot q_{crit}$

Therefore, when large settlements are not acceptable, additional design details must be introduced, to mantain the safety factor:

If stress shall reach q_{crit} there are different solutions available to obtain this result:

1) Grouting of the subsoil.
2) Increasing the depth of retaining wall.
3) Prestressing of the bottom of the pit.

For the first possibility the friction angle has to increase from 35° to 44°.

For the second possibility the depth of wall below the bottom of the excavation has to increase to 6.7m.

For the third possibility it is necessary a prestressing load of 73 kPa.

The subsoil profile is shown in fig. 3 along with the main soil mechanic characteristcs. We can see from this date, that grouting would be very doubtful, even using chemical materials. Because of this peculiar case, the second solution was not practicable and doubtful, because this part could not be braced. The slurry wall could not be extended and consequently the third solution was chosen.

PRESTRESSING OF THE BOTTOM OF THE PIT

The main question for determining the area of prestressing is, up to what distance plastification will occur. O.K. FRÖHLICH deduced this distance.

$$x = \frac{H}{\pi} \cdot \frac{1 - \sin \varphi'}{\sin \varphi'} = 10.7m \qquad (3)$$

from the slurry wall. But this is a very approximate calculation and measurements made on support piles for braces have shown that the main uplift of the bottom of the pit occured at a distance of from 15 to 18m.

The slip surface for foundation failure reaches a distance of 90m, when $\varphi' = 35^o$ and building width (b) equals 14m.

An aproximate computation of the bottom heaving, using logaritimic spirals to represent the yielding surfaces and non-linear shear modulus, shows a bell-shaped displacement of the bottom (fig.1).

Therefore prestressing was performed along all the width of the pit, aproximately 30m.

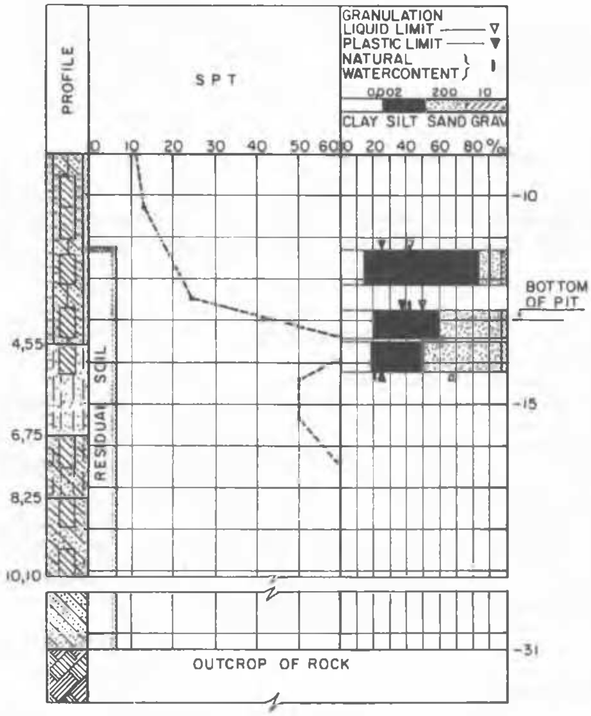

Fig. 3 - Soil Profile and Soil Characteristcs

CONSTRUCTION PROCEDURES

The prestressing of the bottom of the pit was performed, using two different procedures. In one area additional bracing was not necessary and therefore it was possible to excavate small pits (2 x $3m^2$) and to install precast concrete plates of the same dimensions, loading the plate by prestressing of a previously installed tie-back.

Due to the incertainties in the calculations, the tie-backs were anchored 12m below the final excavation level and prestressed with 600 kN, so that the surchage was 100 kPa. (See Selvadurai 1979).

In the second area, this kind of procedure couldn't be used. Excavation of small pits could only have been done by using braced supports. Therefore the method was changed. The precast concrete plate was substituted by a second grouted anchor bulb. Fig. 4 shows the arrangement.

The procedure for this anchor type was:

a) Grout the cable anchor at the lower anchor region.

b) Prestress the anchor with 125% of the final load, using a temporary foundation.

c) Grout the upper anchor region under high pressure.

d) Allow the grout to harden.

e) Release prestressing.

f) Proceed with the excavation, removing the Steel-cable, when necessary, until bottom is reached.

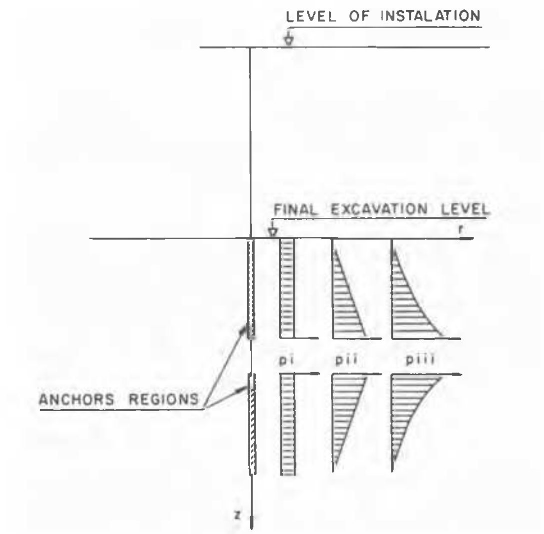

Fig.4 - Layout of double anchored Tie-back installation.

When the double anchored tie-back were released, settlements of the upper anchor region were measured until the hidraulic jack was completely discharged.

Before prestressing of the lower anchor region was begun, creep performance characteristics were verified. The creep factor was $s = 0.16 \times 10^{-3}$m for 125% of final load. Along the axis of the tie-back there was a plastic tube with a bar fixed at the center of the lower anchor region and a external rod fixed at the center of the upper anchor region. No relative deformation between the upper and the lower anchor region was observed during the excavation process.

CONCLUSION AND ACKNOWLEDGMENT

The behavior of the adjacent buildings after prestressing of the bottom of the excavation demonstrated the complete success of the adopted methods.

Figure 5 represents the observation of these buildings. The double anchored tie-backs were just as sucessfull as the anchored precast concrete plates.

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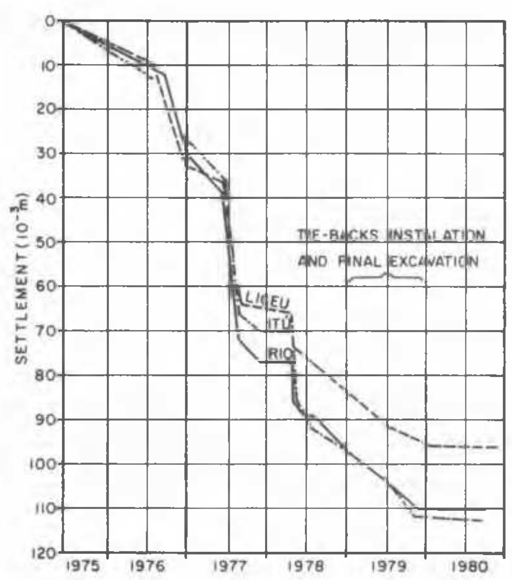

Fig.5 - Evolution of Tie-backs instalation and final excavation.

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