Geotechnical Aspects of Underground Construction in Soft Ground, Mair & Taylor (eds)
© 1996 Balkema, Rotterdam. ISBN 90 5410 856 8

Case history of a ruptured multi tied-back retaining wall

C.E.M. Maffei
Escola Politécnica, University of São Paulo, Brazil

E. Da Silva Guerra & R. L. Leonardo Leite
Companhia do Metropolitano de São Paulo-Metro, Brazil

ABSTRACT: Excavation of a São Paulo Metro Station was performed in residual soil. During excavation a tied-back retaining wall broke off. The rupture is analyzed from a soil-structure point of view. As a consequence of the failure, the construction method was changed to allow catching up with the lost time due to the accident. Reconstruction details are presented as well as re-calculations.

1. INTRODUCTION

The Marechal Deodoro station, in the city of São Paulo, was built by "cut-and-cover" method. Due to the great width and geometry of the underground structure, the temporary works were executed with retaining walls constituted by soldier piles, wood lagging and tie-backs (North, East and West Walls), and with slopes (South Wall), as may be observed in figure 1 and the following figures. It is important to emphasize the proximity of a viaduct.

2. GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS

The local subsoil presents horizons of the Tertiary sediments belonging to the São Paulo Bay constituted by gray and yellow medium to hard sandy clay, located under fine sandy-clayey sediments with yellowish gravel. Superficially the Tertiary is covered by an earth fill 1,0 to 2,0m thick.

The Tertiary sediments are deposited on saprolitic soils constituted of gray and yellow, hard to stiff clayey-sandy silt, originally from Pré-Cambrian granite-gneissic rocks in alteration. Figure 2 shows the subsoil geological profile. For depth greater than 20 m, the SPT value increases significantly, demonstrating stiffer and higher cohesive material, transition from saprolitic to decomposed rock.

The ground water level is located in the Tertiary sediment 2,0 to 3,0 meters deep from the surface, without occurrence of perched water tables. The dewatering was done using deep pumping wells and well points installed on the berms of the South wall. The system was designed to dewater down to the excavation bottom.

3. ACCIDENT OCCURRED ON THE NORTH WALL

Early in the morning of day 13, in July 1985, the North retaining wall failed in an extension of approximately 13 m, in a semi-circle failure surface with radius of approximately 5 m, forming a depression toward the excavation interior. At the time, the excavation was approximately 12 m deep; first and second tie-back levels were already installed, the third tie-back level was not yet pulled. As shown in figure 3, six piles had great displacements toward the excavation area, with failure of the second tie-back level (due to bulb sliding). Even with fully deformed pile, the remaining embedding depth and the lasting resistance of the tie-backs on the first level prevented further damage. The failed part of the wall slid down in 1,5 m and its lower part displaced 3,0m toward the excavation. The wales were extremely deformed or even broken. The soil adjacent to the broken wall slid down 3,0 to 4,0 m, resulting in a vertical slicing surface with a semi-circle shape with radius of 5.0 m. The failure process took about 2 hours to occur, with slow pile motions, only detected at the beginning by noises coming from the wood lagging, then by dropping wedges and surface fissures increasing. Therefore there was enough time to take the equipment out of the excavation area, and to isolate it.

Continuing the failure process, the wood lagging dropped from the piles, and the soil slid inside the excavation up to the point when the tie-back on the second level failed, resulting in sudden and abrupt pile deformations.

Following the wall failing process, and the soil sliding, the situation turned apparently stable, without any signs of any other mayor sliding. Concentric fissures on the surface advanced near 10 m, and at the wall base, a berm with slid material was formed, as shown by figure 3. It must be observed that before the failure occurred, some

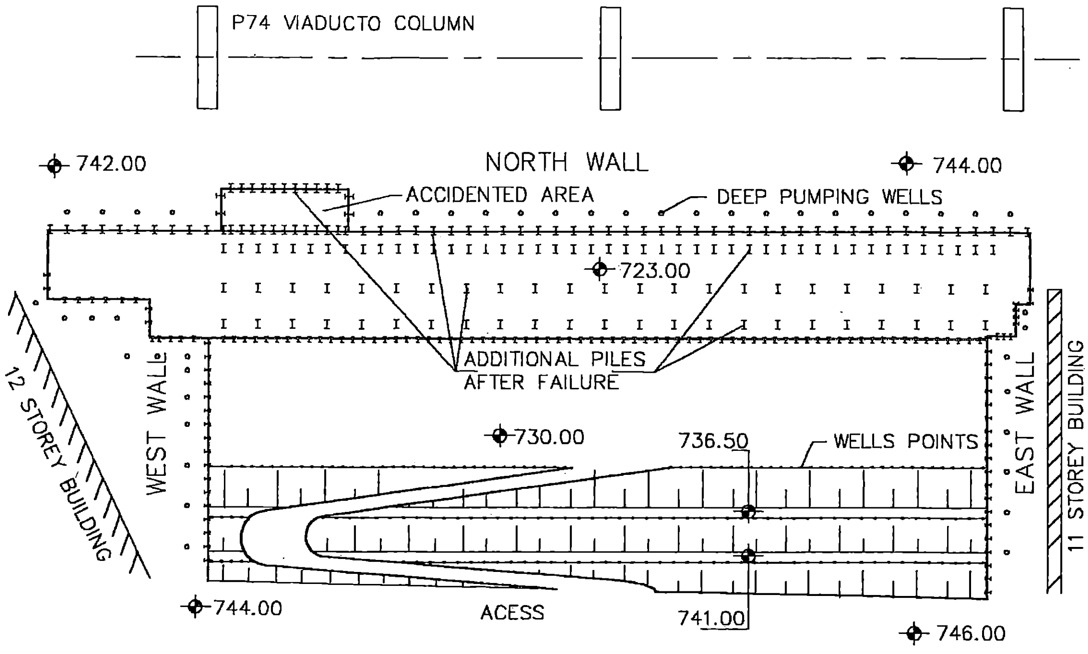

FIG. 1 — EXCAVATION PLAN

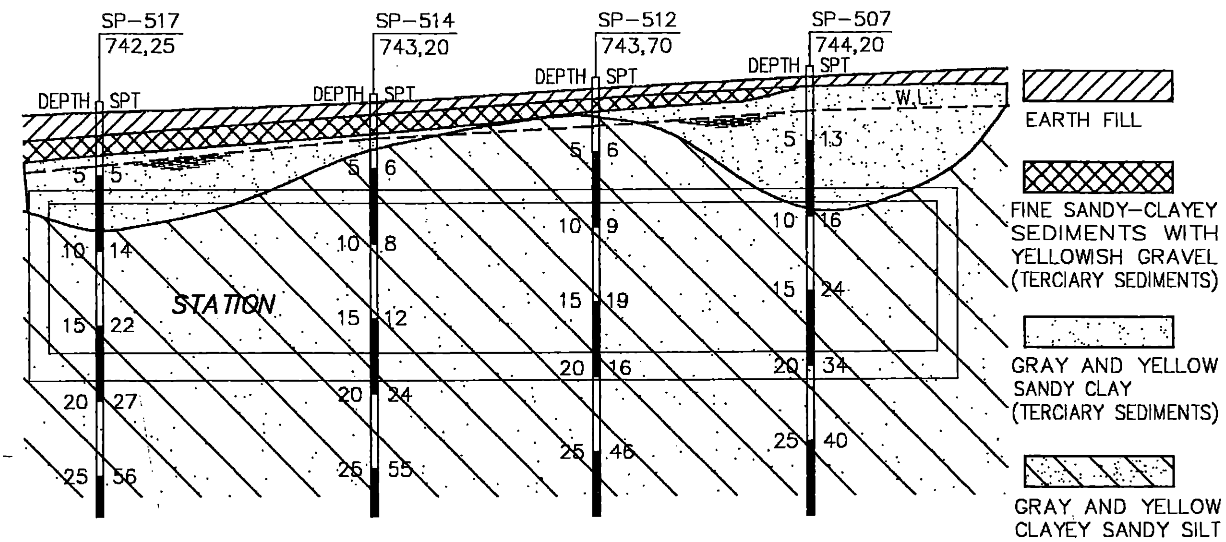

FIG.2 GEOLOGICAL PROFILE

fissures on the pavement, and water flowing through the lagging at the second tie-back level had been noticed.

4. EMERGENCY MEASUREMENTS-1ST PHASE

Despite the apparent stable situation after the failure, there was no assurance that the soil wedge weight failed, but confined, could not overcome the remaining resistance of the ruined wall, causing all the material in the accident area to slide into the excavation. If this process occurred, a progressive failure could start with new sliding surfaces occurring from the fissured pavement, advancing toward the viaduct located from the excavation approximately only 10 m. At that moment, one of the main concerns was with the viaduct column P-74, because the affected area was located near that column. Just after the failure, the first action was to take measurements at the settlement pins installed on that column. It was verified that the column was not affected by the failure.

A berm with 8,0 m high and 5,0 m top width was build in front of the affected area, as shown in figure 4, to reduce risks. A sand filter was placed behind the berm vertically and horizontally under its base to drain emergent water from the lagging, reducing the water buoyancy pressure that could affect the berm stability. The construction of this berm took 45 hours of uninterrupted work.

5. CAUSES FOR THE WALL FAILURE

Some possible causes for the accident were immediately disregarded such as improper execution of the tie-back on the 2^{nd} level, since their load capacity had been previously tested and registered. Also, design error cannot be admitted because the same method and same soil parameters were adopted for the computation of all the 140 m of North wall. The wall displacement and the failure shape suggested a localized problem due to increase in pressures at the tie-backs installed at the 2^{nd} level.

Water seepage was confirmed in the failed slope. Since the piezometer measurements indicated the groundwater below the excavation bottom level, several investigations were made to find the water source. All the public utilities net was checked without success. Only after removing a container located over an electricity box of great dimensions, it was possible to ascertain the seepage cause; the box was full of water. Following the water seepage in the soil it was found a leaking water pipe of 0,25 m diameter and under pressurized water, located about 30 m from the affected area. This same pipe had been shut off before for verifications but the leakage was not noticed because the electricity box worked as a great reservoir with the water slowly leaking into the ground. However, the water seepage was not the only cause. The harmful effect of the water seepage through the residual soil increased the wall pressure by means of the following mechanism: the excavation relieved the confining pressure in the soil causing a reduction of strength in an unfavorable geometry fracture system originating from the parent-rock (caused by the Franki piles installation during the viaduct foundation construction). The water seepage decreased the resistance, causing considerable increase in wall pressure. The wall above, with more pressure on it, and with decreased soil resistance did not keep the soil wedge stable.

6. EMERGENCY MEASUREMENTS -2^{ND} PHASE

Since it was not possible to know if there were unfavorable fracture systems which would allow water infiltration, alongside the North wall, it was decided to extend the berm to all the North wall.

7. CHANGES MADE FOR A NEW CONSTRUCTION METHOD

Two options were possible: 1) to consider the cause for the failure confined only to the failure area; the solution to resume construction would be to treat only the affected area; 2) to consider that all the wall could be subject to the same problem. In this case, changes on the construction method would have to be implemented, to avoid other accidents. Due to the uncertainty, the second option was adopted. The construction sequence was modified, concreting first the slab on the "c" level (731,20) before reaching the excavation bottom. This way it could guarantee safety conditions in a short period of time using the slab as a strut, and saving time that had been lost with the accident. The station construction above and below the "c" slab, progressed as shown in figures 5, 6 and 7. This construction method was called semi-inverted method.

The soil shear strength parameters were reduced as a precaution, obligating reinforcement of the retaining wall. This reinforcement was done by applying one additional level of tie-backs to be installed before the emergency berm removal, and by placing a bracing reinforcement system below the "c" slab level.

8. REDIMENSIONING OF THE RETAINING WALL

The original wall dimensioning was made with the classical semi-empirical method of continuous beam, assuming fixed hinges at the tie-back levels and at the embedment depth zone. The active earth pressure was determined as Rankine and redistributed as shown in figure 8, due to wall flexibility.

The initial soil parameters were adopted c' = 30 kN/m^2 and $\phi'=32^o$ for the residual soil, and were modified for c'= 25 kN/m^2 and $\phi'= 20^0$ after the occurrence of the accident, as a precaution.

Above the "c" slab, that region had the pressure on the wall increased, demanding an additional tie-back level to be installed between the 1^{ST} and the 2^{nd} original levels. Below the "c" slab, it was observed that the 2,26 m embedment depth of pile was not enough for the new increased pressures. Since it was very difficult to reinforce the original soldier pile, this was accomplished by the installation of an additional piling system with double function: to resist the slab weight and other surcharges, and to reinforce the North wall with small additional struts- figures 3, 6 and 9.

The computation method is indicated in figure 9, emphasizing the following aspects : the "springs" which represent the soil in the embedment depth zone have as limit the correspondent passive earth pressure; parametric studies were made to evaluate the influence of the spring coefficient value; the computation method should be evaluative, i.e, it might be considered the excavation history regarding the following cycle : excavation - back filling - installation of the additional tie-back level - excavation - concreting of the "c" slab, etc.; however, it

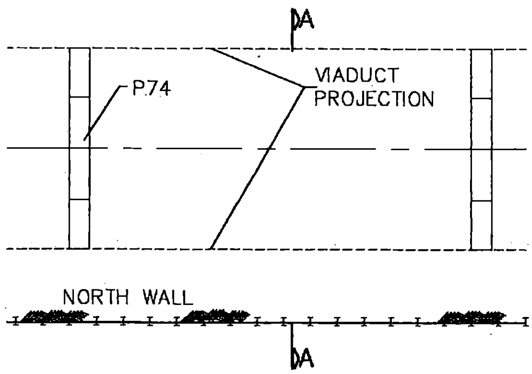

FIG. 3a – PLAN BEFORE ACCIDENT

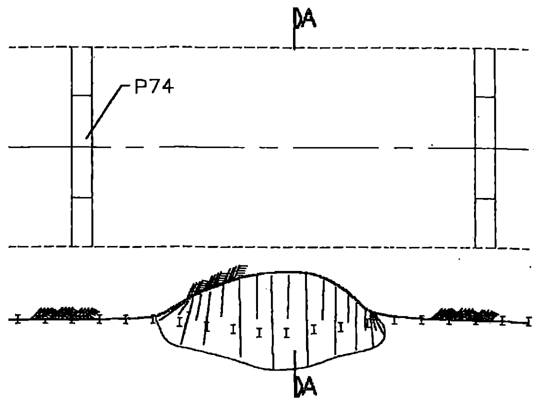

FIG. 3b – PLAN AFTER ACCIDENT

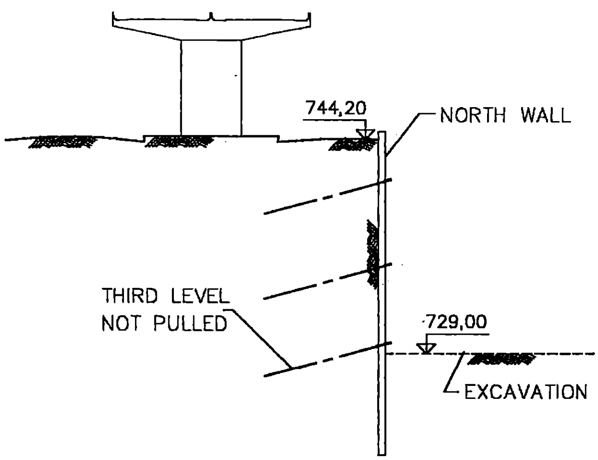

FIG. 3c – SECTION A–A
BEFORE ACCIDENT

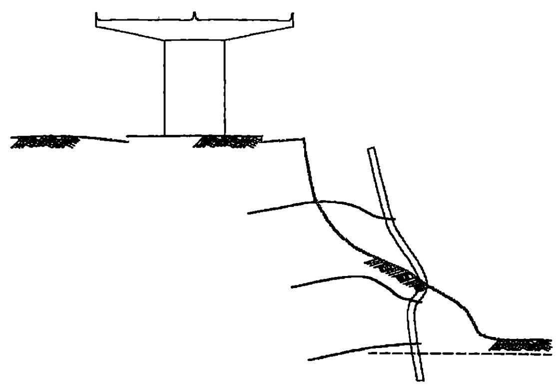

FIG. 3d– SECTION A–A
AFTER ACCIDENT

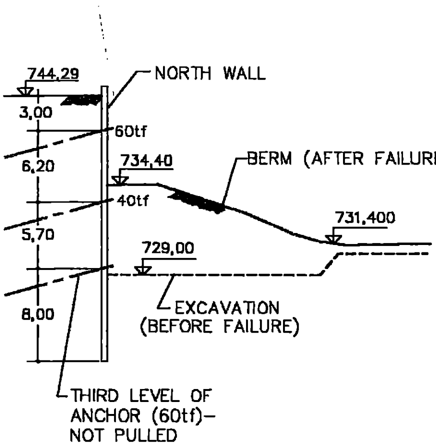

FIG.4
EMERGENCY MEASUREMENTS

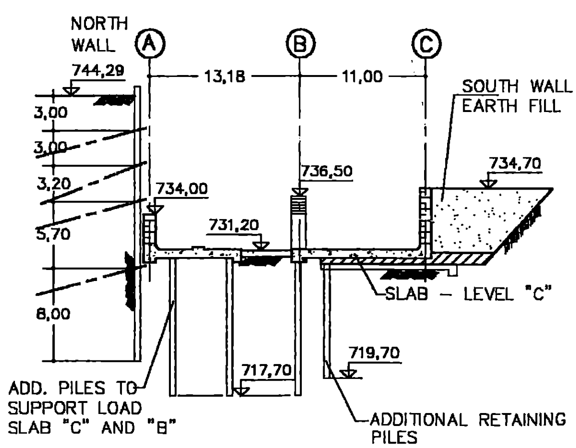

FIG.5
CONCRETING THE "C" SLAB

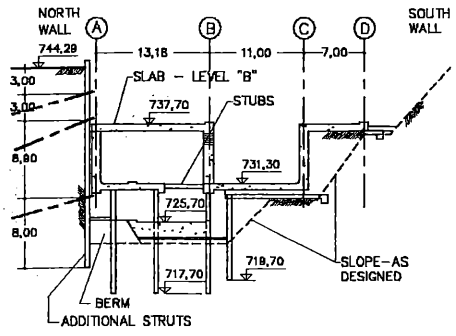

FIG.6
ADDITIONAL STRUTS

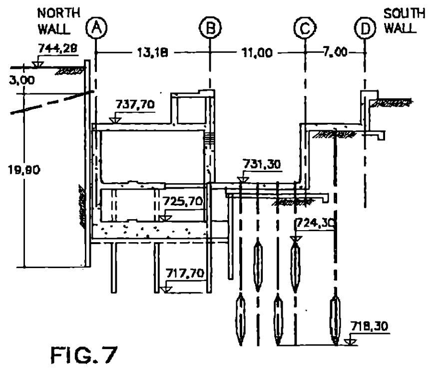

FIG.7
FINAL STRUCTURE

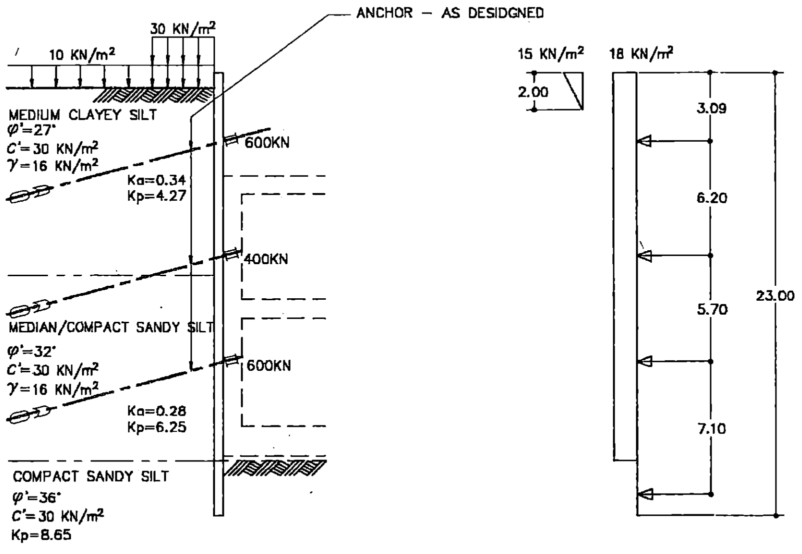

FIG.8
INITIAL CALCULATION METHOD—NORTH WALL

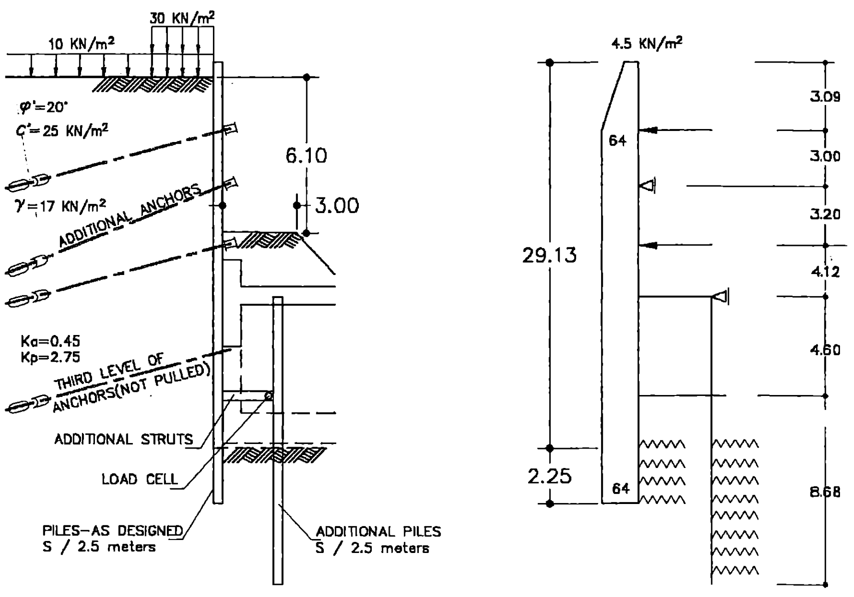

FIG.9
CALCULATION METHOD—NORTH WALL (AFTER FAILURE)

was considered that the non-evolutive method of continuous beam gives a statically admissible bending moment diagram, which is enough from the safety point of view. It must be pointed out that some plastic hinges on piles were assumed.

9. CONSTRUCTION BY THE SEMI-INVERTED METHOD

The basic premises to change the construction method were: to maintain safety conditions, and to ensure two independent reliable work fronts. The first premise was obtained by concreting the "c" slab and part of the South wall, against the North wall (see figure 5) . The second premise became viable through the installation of the three lines of soldier piles which supported, during the excavation under the "c" slab, all the permanent structure weight and the surcharges above the "c" slab level. The construction resumed its activities initially by execution of the additional soldier pile, followed by the "c" slab concrete, with removal, part by part, of the emergency berm. The excavation below the "c" slab level started from the station ends, adjacent to the South wall, facilitating the equipment traffic. The soil was removed by clam-shell. The concrete phase above the "c" level was made independently of the excavation progress below that level.

For the last excavation phase, a berm adjacent to the North wall was left, as indicated in figure 6, with two distinguishing objectives: to install the additional strut level; to observe if the bottom area was drained, because the water seepage could cause decrease of the passive earth pressure in the embedment depth region. The berm was gradually removed to increase the safety conditions, because there was concern about the occurrence of a fracture system in the residual soil distributed in unfavorable condition that might decrease significantly its passive earth pressure value. Also the gradual removal of the berm allowed observation of the loading evolution with the load cells installed on the three additional struts, in front of the affected area. This berm allowed the final excavation level to be reached without stability problems for the embedding depth on the North wall, and allowed confirmation that the excavation bottom was dry. Later on the berm was removed by parts and the bottom slab was concreted. The load cells registered loads in the order of 350 kN, well below the 600 kN predicted on the redimensioning stage. The station structure conception was not altered, making it necessary only to verify the steel reinforcement of some structural elements, mainly the slab at the "c" level to make sure they were compatible with the temporary and final supports. Major difficulties occurred with the semi-inverted method during the concreting of the slab at the "c" level, which used the ground as formwork. Contact of the "c" slab already concreted with the walls under slab poured after, was a weak point from the leakage point of view.

10. CONCLUSION

To use rigid system to retain residual soils is the most important lesson to be learned from rupture. Displacements of walls must be as small as possible to prevent the decrease in strength through unfavorable joint systems.