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REVISING THE CONCEPTS OF THE SOLDIER PILE WALL

REVUE DES CONCEPTES DES PIEUX DE POSEE DEBOUT

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Two projects in Switzerland are presented where innovative designs of soldier pile walls were developed to overcome obstacles and accelerate construction: 1) a definitive retaining structure (27m deep) for an underground parking in a densely urbanized area of Lausanne, where the site was at one time sandstone quarries filled randomly with poor materials overlaying very severely weathered sandstones and shales, 2) a provisory retaining structure (12m deep) for an underground parking in Monthey, adjacent to historical buildings, at a site which is located in the middle of a very wide alluvial fan lateral to the Rhône valley, having sandy silts with stones and crystalline blocks up to several meters in diameter. Site conditions for each site are presented followed by the analyses of the particular constraints relative to the construction of the retaining structures. The development of the conceptual design for each case is discussed as well as some of the alternate designs which were not selected. Construction procedures are explained to show the advantages of the selected designs over the alternate solutions. Numerical techniques used are briefly summarized and the main results of the preconstruction analyses are presented and compared with monitoring results.

1. INTRODUCTION

Soldier pile walls are commonly used all over the world for excavations in cohesive soils with favorable hydraulic conditions. The system relies on vertical elements, the soldier piles, generally steel H-beams driven or set in boreholes of adequate diameter. The excavation procedure begins by inserting between the flanges of the H-beams horizontal elements that constitute the "skin" of the earth retaining structure (ERS), classically made of wood. With the deepening of the excavation, the stability of the ERS is established by incorporating soil anchors of various types, ranging from the "dead soldier type" to the most modern prestressed tendons.

Because of the increasingly tighter construction schedules, more difficult geometrical constraints generated by close proximity to pre-existing structures, and stricter environmental regulations regarding noise and vibrations, innovative designs are being developed as variations on the soldier pile theme.

Two densely urbanized sites where such innovative designs for soldier pile ERS were implemented are described in this paper focusing the attention on the remarkable differences between these designs and more classical solutions. Also, the results of the deformations monitoring of the built structures are presented and compared with the results of the theoretical analyses.

2. CHAUDERON SITE, LAUSANNE, SWITZERLAND

2.1 Geological and Geotechnical Conditions

Boreholes from the site presented the following simplified geological/geotechnical profiles: good quality fills overlay bedrock made of shales (B) and sandstones (C1/C2) in a severely tectonized layered structure. Sandstone layers have thicknesses varying from a few centimeters to a few meters. Laboratory results from mechanical resistance tests (compression σ_c, tension σ_t) are given in Table 1.

Table 1. Compressive σ_c and Tensile σ_t Resistance of Bedrock at Chauderon Site.

Layer	σ_c [MPa]	σ_t [MPa]
B	25 à 40	1.5 à 3
C1	5 à 15	0.5 à 1
C2	>20	1.0 à 3

Only very minimal water flows were noticed within the soil/rock mass.

2.2 Constraints Relative to the Design of the Retaining Structures

The project called for excavation depths of 15m to 27m, including the underpinning of a poorly founded building immediately adjacent to the future excavation. Numerous existing underground structures around the site inhibited the free positioning of anchors, rendering it a very delicate and precise operation. Other obstacles particular to this project included the presence of a bank's central computer at a distance of 20m, necessitating strict vibration control.

2.3 Retaining Structure Concept

Options such as diaphragm walls (too thick) and nailed walls (lack of rigidity) were considered and rejected. In addition, the best time schedule obtained from these options was 7 months before the excavation could be delivered to the structural contractor. A "Down/Up" construction where the excavation is performed under the cover of the slabs simultaneously with the construction of the building was also analyzed and rejected due to the high cost of this method given the particular geological conditions.

It was determined that in the particular geological/urban environment of the project, the best technical/economical compromise would be obtained in combining the underpinning with the construction of the ERS, i.e. setting in place prior to any excavation vertical elements that would assume the function of piles for the pre-existing buildings and simultaneously of soldier piles for the ERS.

In considering the underground obstacles, a rigid skin design was adopted which would then allow easier positioning of the anchors, suppressing the need for a horizontal beam connecting the anchors and distributing the soil thrusts. Also, a rigid skin could serve as a final wall for the underground levels of the proposed building, thus speeding up the global construction schedule.

Regarding drainage, a permanent pumping system was selected since it was the most economical choice given the small amount of seepage.

The main problem to be solved concerned the proximity of the building to be underpinned: The ERS had to be aligned on the outer limit of the façade of this building with the maximum allowable thickness of the ERS at 400-mm. Further difficulties were brought in by aerial obstacles (balconies, other protuberances). The final design considered very compact H-beams of the HEB160 type inserted in the terrain with a "light" machine (150kN overall weight) equipped for down-the-hole hammer perforations of 230mm diameter. The boreholes were grouted in order to keep the H-beams firmly in their position. The horizontal distance between beams was fixed at 2.1m. Because of the very important depth of the ERS (27m), due regard had to be given to deviations of the boreholes and other constructional tolerances. The solution to these points came by fractioning the soldier piles into two halves as shown in Fig. 1.

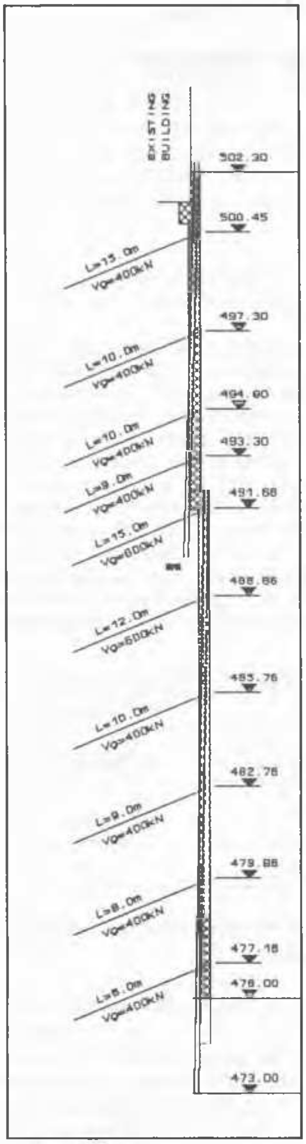

Fig. 1 Typical Cross Section of the ERS at Chauderon.

On the upper portion of the wall, where the fills had to be spanned by the ERS the space between soldier piles was protected by two micropiles reinforced by a 40mm diameter steel bar, as defined in Fig. 2.

Owing to this, the contractor acquired a great freedom of movement. The terrain was sufficiently reinforced to allow the opening of 20-25m lengths by 2.5-3m high panels of "skin" sections to be reinforced and concreted every day, in several concurrent locations along the perimeter of the building.

Finally, the "skin" of the wall was designed as a continuous reinforced concrete plate with a theoretical thickness of 390mm supported during construction by the anchors and, at the final stage, by the slabs of the building.

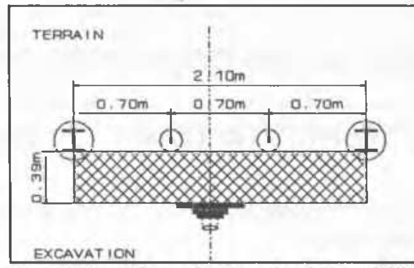

Fig. 2 Plan View of a Standard 2.1m Wide Panel in the Top Half Portion of the ERS (notice the micro-piles).

The horizontal stability of the ERS was obtained by prestressed tendons (400-600kN per tendon) organized in such a way that they would systematically be above the future slabs of the building, thus enabling easy release of the tension and extraction.

The construction was finally completed in 5 months, two months faster than with the best estimate of the other methods.

2.4 Pre- and Postconstruction Analyses

The numerical analyses of the ERS were executed with a specific elasto-plastic software that modelizes the construction phases (Fages and Bouyat, 1971, Rossignol and Aimon, 1980). Hence the algorithm used represents a first step to a refined geomechanical modeling of the situations which arise when dealing with deep excavations. However, in this method, the coupling of the mechanical model to the hydraulic conditions enabled by modern finite elements codes is totally neglected.

Keeping in mind that the rigid concrete "skin" of the ERS was installed immediately after excavation, the computational rigidity of the ERS was assumed to be equal to the rigidity of the soldier piles and the concrete panel. Lateral surcharges, principally derived from the abutting building foundation, were estimated to be 380 $kN \cdot m^{-1}$ over a width of 0.7m, identical to the width of the existing wall.

Due to the varied conditions of rock layers and by field experience, the hypothesis was made that the soil/rock system would behave, when considered at the scale of the ERS, as a soil system with characteristics similar to those of the top soil layer. Therefore the calculation was based on a homogeneous soil-rock system through the depth of the wall with the characteristics described below. Two different sets of soil conditions were analyzed.

Table 2. Geomechanical Parameters for Two Analysis Cases of Chauderon ERS.

Case	γ [kN m-3]	ϕ [°]	K_a [-]	K_0 [-]	K_p [-]	c' [kPa]	E [MPa]
1	20	35	0.27	0.42	3.69	100.0	12.0
2	20	40	0.21	0.36	4.60	50.0	12.0

Ten lines of anchors with cross-sectional areas of $4cm^2$-$6cm^2$, initially loaded with 400kN-600kN respectively, were modelized as elastic elements at horizontal intervals varying from 2.9m-4m and inclined at 20-22 degrees.

Before presenting some results of the analyses it is worthwhile to note that the entire structure was initially analyzed by hand implementing extremely simple models as follows:

- The horizontal average stress under K_0 conditions was determined and subsequently the loading of the soldier piles was determined.
- The bending moments were determined at every excavation stage by considering the soldier piles ends as totally constrained in their rotation by assuming that the top end was already embedded by the concrete panel of the prior excavation stage, and similarly the bottom end by the grout/rock system.
- The socket of the upper soldier piles was primarily dictated by the need of transfering to the bedrock the underpinned loadings. Thus it was finally designed with a length of 2.4m.
- The socket of the lower soldier piles was designed against horizontal thrust using a semi-empirical method leading to a 3m embedment (Dembicky et al., 1977).

These simplified analyses led to astonishingly good results, in excellent agreement with the more refined elasto-plastic procedure, but gave no hints regarding the expected deformations of the ERS.

The reinforced concrete "skin" was designed for the construction stage with a double steel mesh on each face (excavation, terrain, Fig. 2), each mesh giving $3.35cm^2 \cdot m^{-1}$ of reinforcement in both orthogonal directions. The structural engineer in charge of the building checked this reinforcement for the final stage and gave his approval to this reinforcement, the final stage inducing less stresses than the construction stage. The foot of the ERS was designed in order to be able to cope with the permanent loadings induced by the finished building.

The average horizontal stress developed by the anchor system in the top half of the ERS was of $51kN \cdot m^{-2}$, whereas in the midsection, $73kN \cdot m^{-2}$ were introduced by the 600kN anchors. In the lower portion of the ERS the density of the 400kN anchors was reduced so that only $32kN \cdot m^{-2}$ were developed.

Local and global stability were studied with simple rigid-plastic methods and the results were used to define the anchors' free lengths. The theoretically short free lengths of the bottom anchors were increased at design stage in order to avoid accidental overstressing due to winter freezing.

Deformations during construction and after completion were monitored by inclinometers, high precision surveying, anchor pressure cells, and crack monitoring. Vibrations were monitored at the vicinity of the bank's computer. The behavior of the ERS was excellent with deformations being of 15mm in both the vertical and the horizontal direction. The horizontal deformation defined by the elasto-plastic analyses was in excellent agreement with these results.

3. MONTHEY SITE, SWITZERLAND

This project concerns the construction of an underground parking in a place bordered by historical buildings (Fig. 3), with excavation ranging from 11-15m depth.

3.1 Geological and Geotechnical Conditions

Three types of deposits were encountered by the boreholes performed on the site.

Fluvio-glacial sediments form a hill on one side of the site while other boreholes around this hill encountered a finer alluvial deposit. These two different alluvial deposits are underlain by the overconsolidated moraine which was encountered at depths reaching 12m.

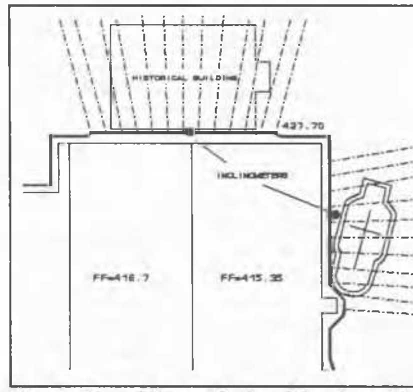

Fig. 3 Plan View of the Monthey Site Showing the Historical Buildings and the Position of Two Inclinometers.

The soil conditions in which the ERS was to be constructed were sandy and silty gravels with numerous stones and blocks, some reaching metric sizes. No signs of water were detected nor expected in this cohesionless soil mass, since the phreatic level lays several meters below the proposed deepest excavation level.

Table 3 presents the design geomechanical parameters at the Monthey site.

Table 3. Geomechanical Parameters for the Analysis of the Monthey ERS.

Case	γ [kN m-3]	ϕ [°]	K_a [-]	K_0 [-]	K_p [-]	c' [kPa]	E [MPa]
1	20	35	0.27	0.42	3.69	0.0	15.0

3.2 Constraints Relative to the Design of the Retaining Structures

The proximity of roads as well as historical and other buildings led to the necessity of vertical cuts. Several alternate designs were carefully examined, but the presence of large crystalline blocks made it clear that the feasibility was dramatically linked to the capabilities of the candidate technique to cross these stochastic obstacles.

Finally, as explained in the next section, a solution similar to the one presented for the Chauderon, Lausanne site was selected, but duly modified in order to cope with other geotechnical conditions, smaller depths, and different imposed construction criteria, i.e., provisory retaining structure.

3.3 Retaining Structures Concept

All faces without adjacent historical buildings were equipped with nailed walls following a classical constructional scheme originally developed in Germany. At the critical faces, an innovative variation of the soldier pile wall was set in place in order to allow simultaneous underpinning, control settlements, and enable faster work under reasonable safety conditions relative to noise and vibrations.

The soldier piles were designed as H-beams of the HEB-220 type, set into the ground prior to excavation by boring 300mm diam. boreholes with a down-the-hole hammer. The beams were then stabilized by a multirow anchor system (Fig. 4), with 400kN prestressed tendons to be set in service in parallel with the progress of the excavation.

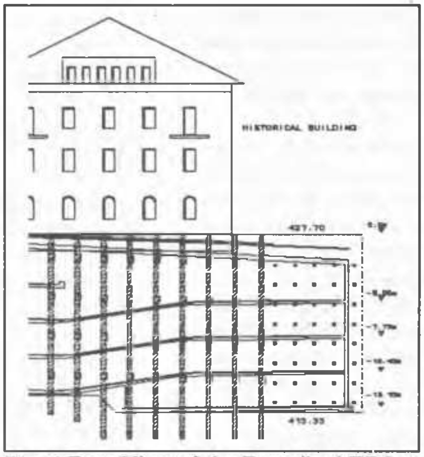

Fig. 4 Face View of the Described ERS at Monthey Showing the Transition Between Soldier Pile Wall and Nailed Wall.

Finally, the "skin" of the ERS was a reinforced gunite membrane that was connected to the front of the H-beams instead of being inserted behind the flanges as would be normally done following a classical constructional scheme. The connection was obtained by welding specially designed reinforcement bars (16mm diam., 200mm intervals) to the flanges of the H-beams: through this technique immediate support was given to the very poorly cohesive soils, avoiding loss of material and expediting the gunite process. The anchor forces were transmitted to the system by carefully designed steel plates, each anchor being set on the side of the H-beam it was to stabilize.

3.4 Pre- and Postconstruction Analysis

The 2m spacing between soldier piles was selected using simple numerical techniques as described in section 2.4. The rigidity of the system and its flexural resistance during construction stages permitted a distance of 3m between the anchor rows.

Again, the sockets of the piles at the bottom of the excavation were defined primarily by pile allowable resistance considerations (Poulos & Davis, 1980, Berezantev, 1961, Oboni, 1988), leading to 2m sockets, then the Dembicky (1977) method was used to verify the horizontal stability.

The gunite was analyzed as a membrane using the Timoschenko-Boobnov method (Timoshenko, 1940) as follows. The 150mm thick gunite sheet, reinforced with a $3.35 cm^2 \cdot m^{-1}$ steel mesh is transformed into a "steel equivalent membrane" submitted to a spread uniform loading resulting in a membrane tensile effort as well as a residue bending moment. This bending moment represents only 11% of the moment that would be generated if the gunite was rigid enough to act as a slab, i.e. if no tensile effort was generated. The maximum resultant stress of the "steel equivalent membrane" was evaluated at $\sigma = 164 N \cdot mm^{-2}$ which is perfectly compatible with the steel used in the reinforcing mesh.

The maximum horizontal deformation measured by the inclinometers reached 12mm, and was accompanied by vertical settlements less than 5mm. This deformation was higher than the one computed with the elasto-plastic method.

CONCLUSIONS

Two examples located in western Switzerland are presented where innovative variations of the well known soldier pile wall have been implemented. The main advantages of these innovative techniques are brought by the small size of the soldier pile that can be set into predrilled boreholes performed by light machines, capable of working under limited height.

Also, a careful study of the local conditions can lead to new choices relative to the construction of the "skin" of the ERS: two opposite extremes have been shown in this paper in order to explicitly address a provisory case and a definitive case.

The computational aspect has been addressed, showing that simple but conceptually clear partial models can lead to results that are in good agreement with more sophisticated methods. Also, when comparing the results of the elasto-plastic analyses with the monitoring results, good agreement has been found in the cases described in this paper.

Finally, regardless of the theoretical modeling of earth retaining structures, it is essential that, to obtain good limitation of deformations, attentive surveillance by the designer is carried out during all construction phases. This applies particularly to the construction sequence of the skin, the size of panels opened, and the progression of tensioning the anchors.

REFERENCES

Berezantev V.G., Khristoforov V., Golubkov, V., Load Bearing Capacity and Deformation of Piled Foundations, *Proc. of 5th Inter. Conf. on Soil Mech. and Found. Eng.*, Vol 2, 1961.

Dembicky E., Odrobinski W., Cichy W., Stabilité des Fondations des Poteaux Soumis à des Moments, *Annales de l'Institut Technique du Bâtiment et des Travaux Publics*, 1977.

Fages R., Bouyat C., Modèles Mathématiques Intégrant le Comportement Irréversible du Sol en Etat Elastoplastique, *Travaux*, 1971.

Poulos H.G., Davis E.H., *Pile Foundation Analysis and Design*, John Wiley & Sons, 1980.

Oboni F., Evaluation Probabiliste des Performances des Pieux Forés Chargés Axialement en Tête, *Thèse 716*, Ecole Polytechnique Fédérale de Lausanne, EPFL, 1988.

Rossignol Ph., Aimon M.-J., Application aux Calculs sur Ordinateur des Rideaux, des Plaques et des Pieux, *Annales de l'Institut Technique du Bâtiment et des Travaux Publics*, 1980.

Timoshenko S., *Theory of Plates and Shells*, Van Nostrand Reinhold, New York, 1940.